Environment Canada | Environnement Canada

Canada - Ontario Agreement on Great Lakes Water Quality

Ministry of the Environment
Ontario

Storm Water Management Model Study

Volume I

Research Program for the Abatement of Municipal Pollution
under Provisions of the Canada-Ontario Agreement
on Great Lakes Water Quality

STORM WATER MANAGEMENT MODEL STUDY

VOLUME I

by

Proctor and Redfern Limited
and
James F. MacLaren Limited
Toronto, Ontario, Canada

MARCH 1976

Sponsored by the Urban Drainage Subcommittee
Canada-Ontario Agreement

COA Project No. 73-5-10
DSS Contract Serial No. OSS4-0046

This document may be obtained from -

Training and Technology Transfer
Division (Water)
Environmental Protection Service
Environment Canada
Ottawa, Ontario
K1A 0H3

Ontario Ministry of the
Environment
Pollution Control Branch
135 St. Clair Avenue West
Toronto, Ontario
M4V 1P5

TABLE OF CONTENTS

74477

Page

LIST OF FIGURES

SUMMARY

SUMMARY

This report documents the findings and conclusions of a study for the development of a Storm Water Management Model. The Terms of Reference for this study were established by Environment Canada and the Ontario Ministry of the Environment under Contract OSS4-0046 dated May 30, 1974. Scientific Authority and Liaison were vested in Mr. J. Marsalek of the Canada Centre for Inland Waters, Mr. G. Mills of the Ministry of the Environment, and Dr. R. Slater of Environment Canada. The study was conducted jointly by Proctor & Redfern Limited and James F. MacLaren Limited.

The objectives of the study were:

(a) To review and modify the U.S. Environmental Protection Agency Storm Water Management Model (SWMM) for Canadian Conditions.

(b) To select and modify a submodel for the simulation of quantity and quality of runoff resulting from snowmelt.

(c) To develop a Data Analysis Submodel capable of extracting the SWMM meteorological data input from existing data banks.

(d) To select and modify a high speed model for simulation of frequency of discharges and overflows, with the capability to use output from the Data Analysis Submodel as input.

The report documentation of this study is contained in four volumes: Final Report (Volume I), Technical Appendices (Volume II), User's Manual (Volume III), and Program Listings (Volume IV).

This initial "Summary" chapter presents a concise synopsis of all subsequent chapters in Volume I, and serves as an introduction to the study.

PART I: INTRODUCTION

CHAPTER 1 - Basic Concepts

Several factors contribute to the need to review traditional methodologies employed in municipal drainage design and pollution control:

(a) Concerns over new developments resulting in increased flood peaks and extensive downstream flooding.

(b) Measurements have shown that the concentrations of suspended solids in surface runoff may exceed those recorded for sanitary sewage, while B.O.D. levels are generally similar to those in the effluent from a secondary treatment facility. Consequently, the effects of storm runoff must be considered in the overall cost benefit analysis when pollution control measures, such as combined sewer separation or the upgrading of treatment plants, are considered. In some cases, the storage and treatment of urban runoff may be more cost effective than other pollution control measures.

(c) Urban runoff models have been developed with the capability of simulating the quantity and quality of stormwater flows. These models generate overland flow hydrographs for subwatersheds. The accumulation, and washoff, of surface pollutants is computed, and the resulting hydrographs and pollutographs are routed through a schematic representation of the actual or proposed sewer system. Because these models operate with hydrographs, and not merely with peak flows, the effects of storage may be considered at various surface and subsurface locations. Some models also are capable of simulating the the effect of different treatment processes on the storm or combined flows, and subsequently, assessing the effects of discharges and overflows in the receiving water.

CHAPTER 2 - Scope of the Study

Engineers intending to use modelling techniques face the task of selecting the most appropriate model for the particular phase of the work under consideration. Having selected a good model, it is necessary to apply it in a useful and sensible manner, bearing in mind its inherent limitations. This study administered by the Storm and Combined Sewers Sub-Committee within the framework of the Canada-Ontario Agreement, was required to make available a package of models suitable for a wide range of applications in Canadian conditions.

An analysis of different urban runoff models and a review of previous studies in Canada, the U.S. and Europe, indicated that one of the most comprehensive and best documented one-event simulation models is the Storm Water Management Model (SWMM) developed for the U.S. Environmental Protection Agency. The Terms of Reference of this study required that this model be rendered fully operational and subsequently calibrated using the results of measurements recorded in various Canadian municipalities. The sensitivity of the model to various input parameters was to be investigated and methods of aggregating various input data in order to simplify data preparation and reduce computer processing time were to be proposed.

The routing or TRANSPORT routine in the SWMM was to be assessed in comparison with the more sophisticated computations of the Water Resources Engineers (WRE) and Dorsch Hydrograph Volume Method (HVM) models, and the appropriate model for particular types of analysis identified.

It was also required to revise the Storage and Treatment Block of the SWMM in order to better reflect Canadian conditions.

The other routines of the model, "Inflow-Infiltration" and "Receiving Water Block", were to be assessed on the basis of literature reviews and general applicability.

The importance of snowmelt in the simulation of the quantity and quality of runoff was recognized as being of particular importance in Canada. The SWMM model did not consider snowmelt, and so it was necessary to select a snowmelt routine for integration with the SWMM.

A data analysis model, capable of processing meteorological data directly from the Atmospheric Environment Service data bank into a form suitable for input to various urban runoff models was also developed.

The "STORM" model developed by the U.S. Corps of Engineers, was selected for the simulation of frequencies and magnitudes of overflows, and tested on different watersheds.

The research was carried out according to detailed Terms of Reference and the results presented in the four volumes of this report.

Volume 1	Final Report - main findings and conclusions of the study
Volume 2	Technical Appendices - detailed literature reviews and additional data
Volume 3	User's Manual - integrated with the equivalent EPA publication
Volume 4	Program Listings - SWMM, STORM, Generalized quality model, data processing program

The individual chapters of the Final Report are summarized in this synopsis.

PART II: APPLICATION OF SWMM ON CANADIAN WATERSHEDS

CHAPTER 3 - Assessment and Selection of Canadian Watersheds

An initial review of the data requirements of urban runoff models and available measurements from urban watersheds in the U.S. and Canada revealed that, at the time of the study, the existing data from any one watershed was not sufficient for all model testing purposes. Since there were very few fully integrated sets of measurements of quantity and quality and rainfall data, it was necessary to employ data originating from different areas for different aspects of the studies. The adequacy of data from Canadian watersheds was found to be comparable with that from U.S. watersheds. Canadian drainage areas simulated were:

(a) The 542 acre Bannatyne combined sewer district in Winnipeg,

(b) The 2330 acre West Toronto combined sewer district,

(c) The 48 acre Brucewood separate sewer district in North York, Toronto.

There were data voids in all areas, but a certain number of events were isolated, for which there was reasonable confidence in the measurements. The report contains detailed descriptions of these study areas and includes a discussion of possible sources of error. More recent data acquisition efforts have also been briefly reported.

CHAPTER 4 - SWMM Flow Simulations for Selected Canadian Watersheds

A review of previous studies in Canada and other countries indicated that SWMM flow simulations compare reasonably well with measurements and the results of other urban runoff models. The only watershed previously used for model testing in Canada was about 100 acres in size. Studies presented in the report for areas ranging in size from 48 to 2330 acres indicate that about two thirds of measured peak flows may be simulated by SWMM within $\pm$ 20 per cent .

A sensitivity analysis of the various parameters and default values confirms the suitability of the program defaults in most applications.

In the case of a system under surcharge, this work indicates that the shape of the SWMM computed hydrograph is distorted. Surcharged conditions usually occur fairly infrequently and so this weakness in the model will not be particularly significant in studies for pollution abatement. However, a more comprehensive analysis is required during the design of new drainage or relief sewers.

Simulations of surcharged systems conducted using the HVM and WRE models indicate a superior simulation compared to the original SWMM simulations. Since inlet hydrographs generated by the three models are very similar, the improvement resulted from the more sophisticated pipe routing techniques employed in the HVM and WRE models. The models were compared on the basis of the underlying theory as well as data and computer processing requirements. The performance of the WRE model was found to be equivalent to that of the HVM. According to information from EPA, an updated version of the WRE model will be incorporated in a new 1976 version of the SWMM, as well as the snowmelt routine developed in this report.

The principal conclusion is that the most recent version of the SWMM is an appropriate tool for a broad range of studies, from preliminary planning to practical design work (with the exception of design and analysis of systems under surcharge). Further field measurements for calibration of the model solely for flow simulation may only be required in special cases.

CHAPTER 5 - SWMM Quality Simulations for Selected Canadian Watersheds

A comprehensive review of quality modelling and available field data shows that the simulation of the quality of storm related flows is not as accurate as flow simulation. The simulations of storm water quality conducted for the Brucewood watershed and for other areas, emphasize that calibration with measurements is required for an "order of magnitude" agreement. In the Bannatyne combined sewer district, where sewer grades are very flat, the scouring of extensive depositions in the trunk lines is a major factor affecting the quality of the combined overflow. These phenomena are reasonably well simulated by the SWMM.

The two optional methods included in the SWMM for suspended solids computations were compared and, while neither seems to be comprehensive, some recommendations for the use of the appropriate method in different circumstances are given.

Sensitivity analyses were conducted for variations in the input parameters describing pollutant accumulation, such as the length of the antecedent dry period, cleaning frequency, specific gravity of sediments, washoff and accumulation rates,and sewer grade. The basic algorithm used in the SWMM quality simulation appears to account logically for these variations. It seems that the present state of the art in this area does not justify sophisticated pollutant routing or the consideration of pollutant decay while in transit. It appears therefore that calibration may be facilitated by a more simple approach as described in Chapter 10.

It is concluded that since the model responds in a reasonable manner to alternative input conditions, it may be used within the documented limitations to compare various control alternative.

CHAPTER 6 - Assessment of Additional SWMM Routines

A review of some Canadian data and of the methodology employed in the SWMM infiltration sub-routine highlights the need for additional infiltration measurements. The application of SWMM to the study of sanitary sewer inflow-infiltration is briefly discussed.

The SWMM Receiving Water Block was debugged and applied in several hypothetical studies. A theoretical comparison with more sophisticated models is presented. The model appears to be most useful for planning applications, in which different alternatives have to be assessed under conditions of limited data.

PART III: MODIFICATIONS TO SWMM AND AUXILIARY ROUTINES

CHAPTER 7 - Storage/Treatment Routines

A review of the literature regarding treatment of storm runoff and overflows is presented in Volume 2 of the report. Following an analysis of the model and the literature review, several changes in the treatment model have been made to reflect actual operating practices. Modifications have been included in by-passing of flows, modelling of solids settling in

storage, and the underflow in the swirl concentrator. The revised model provides a sub-routine designed to estimate the cost of treatment units based on the Canadian market.

Each treatment unit in the model was tested separately and the performance of a combination of storage and treatment units for a particular storm is illustrated. The limitations and potential uses of the model are discussed.

CHAPTER 8 - Snowmelt Quantity and Quality Routines

Selection of a relatively simple snowmelt model resulted from an extensive review of the literature, presented in Volume 2. The model was modified and integrated into SWMM and is now available as a user option. Required input includes hourly temperature and wind velocity information and a description of ground snow distribution and equivalent water depth. Preliminary applications of the model using published measurements and the limited amount of data available for the Brucewood district indicate satisfactory operation.

The available literature describing the accumulation of contaminants in the snow layer and, in particular, their pollution of the snowmelt runoff is rather limited. At this stage, an equation similar to that used in the model for surface runoff is recommended for the estimation of lead and chloride concentrations in the runoff from snowmelt events. Comparisons with measurements in Brucewood show that the model can be calibrated if data are available.

CHAPTER 9 - A Simplified Method for SWMM Modelling-Aggregated Catchments

Simplified procedures leading to reductions in computer processing time and data collection requirements were developed. The essential problem when employing less detail in modelling an area is that more conduits are omitted from the analysis, and consequently less conduit storage is available in routing of the storm flows. Compensating surface storage may be

introduced to make up for the lost conduit storage, and the characteristics of the outflow hydrographs may be preserved even when very coarse discretizations are employed.

These essential concepts are tested on four areas; West Toronto, Bannatyne, and two hypothetical areas of different sizes. The results of these applications for both quantity and quality of runoff compare very well with those of the equivalent detailed simulations. Also described are a number of simulations based on larger time steps for the rainfall input and flow computations.

The methods recommended for the aggregation of subcatchments significantly reduce the costs of data preparation and computer processing time.

CHAPTER 10 - A Generalized SWMM Quality Model

The basic concepts employed are similar to those used in the SWMM. However, this new model has been formulated in such a way as to permit any input hydrograph to form the basis for quality computations. The input flows can originate from measurements or any urban runoff model. This generalized quality model does not route pollutants through a pipe system and consequently requires significantly less computer time than a model which includes routing. The independent calibration of all quality parameters is permitted, so that the model may be rapidly adjusted to local conditions. An overall description of the model is included as well as some verification of the model using the Brucewood data. The results are similar to those obtained using the more complex SWMM.

PART IV: INTERFACING SWMM WITH CONTINUOUS SIMULATION MODELS

CHAPTER 11 - Continuous Simulation Models

A review of the available continuous long term simulation models led to the selection of STORM for this purpose. The main reasons for this choice were the widespread acceptance and simplicity of the model. STORM was applied for the West Toronto and Bannatyne areas. The occurrence of

overflows simulated by the model agrees well with overflow records in these districts. The volumes of overflow predicted agree well with measurements and those computed by the SWMM. The model should be used in the planning stage of a project.

The use of the model for assessing the effects of urbanization is briefly discussed and the use of more recent values for rural pollutant accumulation is examined in Volume 2. The potential for application of the SWMM as a quasi-continuous model is also explored. The use of a simplified SWMM for the analysis of a considerable number of significant events would permit a more complete evaluation of storage and treatment than allowed in STORM.

The interfacing of the Data Analysis Programme (DAP), STORM and SWMM in a planning study is illustrated by means of a hypothetical test case. STORM is used for a preliminary screening of the events and for critical period identification. The detailed simulation of individual events is conducted using the SWMM. Further refinements in an actual study of an existing system might involve the use of the WRE routing routine for the analysis of surcharge.

CHAPTER 12 - Development of Meteorological Data Analysis and Processing Programme

The large volume of meteorological data required for continuous simulation makes computer processing of the input data highly desirable. Consequently a data analysis programme was developed to read hourly precipitation and daily maximum and minimum temperatures directly from the data bank of the Atmospheric Environment Service. The model also screens the precipitation and temperature data for errors and constructs a complete record for each station, employing data from different sources to fill data voids. The precipitation records of up to six stations can be combined, and the computed precipitation record tested for consistency. Finally, the processed data is punched out in a format suitable for direct input to STORM. Short interval rainfall data can also be read and processed.

CHAPTER 13 - General Conclusions and Recommendations

The comparison of the SWMM with measurements for several sewer districts, as well as a detailed consideration of the theoretical basis of the model indicates that it simulates flow quantities with a satisfactory accuracy, over a wide range of conditions. The use of the model for flow quantity simulation may therefore be recommended for planning and design studies related to all aspects of stormwater management.

At the present time, the SWMM simulation of flow quality may also be used for evaluation of the pollutional load associated with urban runoff and combined sewer overflows. If used within its limitations the model is a reasonable tool for comparison of alternatives for pollution abatement, including comparison of the costs of different combinations of storage and treatment facilities, and the effects of discharges in the receiving water.

Although additional flow measurements are no longer required for the substantiation of SWMM flow simulation routines, quality measurements will generally be required for the calibration of the quality routines in the design phase of a pollution abatement study. Until a comprehensive data bank of reliable quality measurements is established, it will not be possible to simulate pollutographs with a consistently good accuracy.

The study has also indicated that the number and quantity of overflows may be approximately evaluated by the simple STORM model. This model should be used in the planning phase of a study for an indication of the changes in runoff quality associated with various alternative developments.

This study has expanded the capabilities of the original SWMM model. Methods of aggregating the properties of individual subcatchments into a single equivalent catchment can lead to a considerable reduction in data preparation and computer processing time. A generalized quality model has been developed, based on the SWMM equations and employing the simplifications obtained using the aggregating techniques. Snowmelt quantity and quality models have been integrated with the SWMM. Methods of inter-

facing the SWMM and STORM models in a planning study have been discussed and demonstrated and a fully operational package of models capable of simulating all aspects of stormwater flow and quality is now available for application in Canadian conditions.

Additional improvements to the SWMM and STORM, and new areas for their use in applications such as the simulation of inflow into sanitary sewers, continuous simulation and rural hydrology, have been suggested. It is probable that both models will continue to be improved due to their widespread use and in particular because of the efforts of the EPA and U.S. Army Corps of Engineers. Analysis and discussion of other urban runoff models indicates that there are a number of adequate tools available and these will probably be used in parallel to the SWMM and STORM. Modellers with experience in the application of the SWMM and STORM will have a firm grounding in the use of urban runoff models and could rapidly become familiar with other models.

Although further research should be carried out in the areas of model improvement and data collection, it is considered that one of the most productive areas for new research is the investigation, by means of the models, of stormwater management methods to be implemented in practical applications.

BASIC CONCEPTS OF STORM WATER MANAGEMENT MODELLING

CHAPTER 1

CHAPTER 1
BASIC CONCEPTS OF STORM WATER MANAGEMENT MODELLING

1.1 GENERAL

The expenditures for drainage works and pollution control facilities are among the largest items in the budgets of most municipalities, and represent a significant percentage of Federal and Provincial funding of public works. Design and planning procedures firmly based on the fundamental processes governing the quantity and quality of urban runoff flows will result in the most cost effective solutions to the problems facing planners and decision makers in most urban areas.

The widespread access to large computers and the recent instigation of sampling programmes have led to the development of urban runoff models, calibrated and validated by comparisons with field data. Consequently, there is a definite trend towards the replacement of the empirical methods of the last century, such as the Rational Method, which are still predominant in Canada, by a more advanced design methodology.

The Canadian Urban Drainage Programme is administered by the Storm and Combined Sewers Subcommittee working in the frame of the Canada-Ontario agreement with the following principal objectives [1]:

(a) to define the magnitude of pollution due to stormwater in the Great Lakes basin.

(b) to establish priorities and schedules for studies directed towards potential solutions.

(c) to develop a strategy for implementing solutions.

The scope of the present study, which forms one of the parts of this programme, is to make available to Canadian Practice a package of models, tested on typical Canadian urban watersheds. These must be capable of simulating urban runoff under the wide range of meteorological conditions in Canada. The two principal models analyzed in this study are the EPA-SWMM (Storm Water Management Model) and the U.S. Army Corps of Engineers' STORM [2,3].

This chapter is a review of the basic concepts of modelling. The two principal models are also briefly described.

1.2 THE NEED FOR MODELS - RECENT APPLICATIONS

The need for a comprehensive approach to the simulation of flow quantities and the limitations of the Rational Method have been recognized since this method was proposed at the end of the last century. For urban areas, however the gradual replacement of this method by methods based on hydrological principles only began in the late 1950's. Hydrograph methods had been introduced much earlier in rural hydrology. The principal reasons for the time lag appear to be the lack of rainfall and flow measurements and the fact that expenditures for the installation of storm sewers and culverts were, in the past, less significant than in other areas of water resources.

The first uses of hydrologic models for urban flow simulation followed the development of the RRL Model (Road Research Laboratory) in the U.K. and the Chicago Model in the U.S. [4,5]. The Dorsch HVM model has since received widespread acceptance in West Germany [6]. Many models have been developed in the U.S., such as the EPA-SWMM, the WRE model, the University of Cincinnatti model, ILLUDAS, MIT, HYDROCOMP etc. and are described in recent literature [7]. Other models such as the QUURM are being developed by Canadian Universities [8].

Because of the multiplicity of urban runoff models available, several comparative studies have been published, in Australia [11], the U.S. [12], and recently in Canada [13]. The Canadian study indicated that for watersheds of less than 100 acres a number of models such as the RUNOFF block of the SWMM, Dorsch HVM surface routine, RRL, UCUR, QUURM can simulate approximately two thirds of measured peak flows to within ± 20 per cent. Larger discrepancies of ± 50 per cent were found with the Rational Method.

Experience gained via practical applications indicates that there are more advantages to hydrograph modelling techniques, as compared to the traditional approach, than merely increased precision. The new methods allow for the simulation of storage, backwater effects, inter-connections, and provide the practitioner with a means of developing a more imaginative and predictable design at a considerable saving [15].

In Canada the first large scale model applications were conducted in Toronto for the analysis of relief sewers by the HVM-Dorsch method. Subsequently, in Winnipeg, relief sewer design and other drainage studies by the SWMM and WRE models resulted in significant savings and the City of Winnipeg now recommends the more advanced methods in its Drainage Criteria Manual. Simulation models are also being applied for Edmonton, Mississauga, Nepean Township, Ottawa, Hamilton, Vaughan, Vancouver and at the Toronto International Airport [14].

The increasing interest in modelling is evidenced by the large attendances at modelling seminars and courses organized by the EPA [9]. SWMM applications have been conducted in Chicago, Philadelphia, Boston, San Francisco and elsewhere. Recently the City of San Francisco has published a User's Manual for the San Francisco (WRE) Model [10].

The simulation of storm water quality together with the computation of flow quantities has started more recently, largely due to the realization that storm sewer discharges constitute a significant pollutional load on the receiving waters. Suspended solids flushed from streets by stormwater may

be more concentrated than those in sanitary wastes, while B.O.D. levels are often similar to those in the effluent from secondary treatment plants. Another problem of concern is pollutional load from combined sewers. It is estimated that one half of the urban population of Canada is at present served by combined sewers. The runoff flow generated from moderate rainstorms of about 0.1 inches per hour can be considerably in excess of the amount of "three times dry weather flow" that is normally intercepted and treated at a municipal treatment plant. Consequently, combined sewage is frequently overflowed to the receiving water. The shock loads imposed on the receiving water body can result in short term hazards to health and general environmental degredation over a longer period.

Both quantity and quality simulations are needed for the adequate design of drainage in new developments. Traditional drainage systems usually result in an increase in peak runoff flows, compared with the non-urbanized situation. Often upstream development can cause downstream flooding, and an additional pollutant load to the stream. The traditional drainage design approach has been unable to deal with such problems, principally because the analysis of storage requires a consideration of the flow-time relationship, which is the basis of all hydrograph models.

Today, decision makers in the field of pollution control are faced with several recurrent questions, which cannot be adequately answered without a comprehensive analysis. These include:

- How should runoff from new developments be controlled?
- How is the flow and water quality in the receiving water affected by the various alternative drainage schemes that might be employed in new urban areas?
- Is sewer separation or construction of road sewers economically and/or environmentally justified as compared with other options?
- How can upstream storage control be balanced with in-system storage?

- How can a balance be achieved between cleaning operations and downstream treatment practices?
- What are the optimum levels of treatment and storage?
- Is advanced treatment of sanitary flows as cost effective as the control of urban runoff?

Some of these and other problems could be clarified as a result of extensive monitoring programmes. However, these programmes are not always effective and can be very costly when conducted properly.

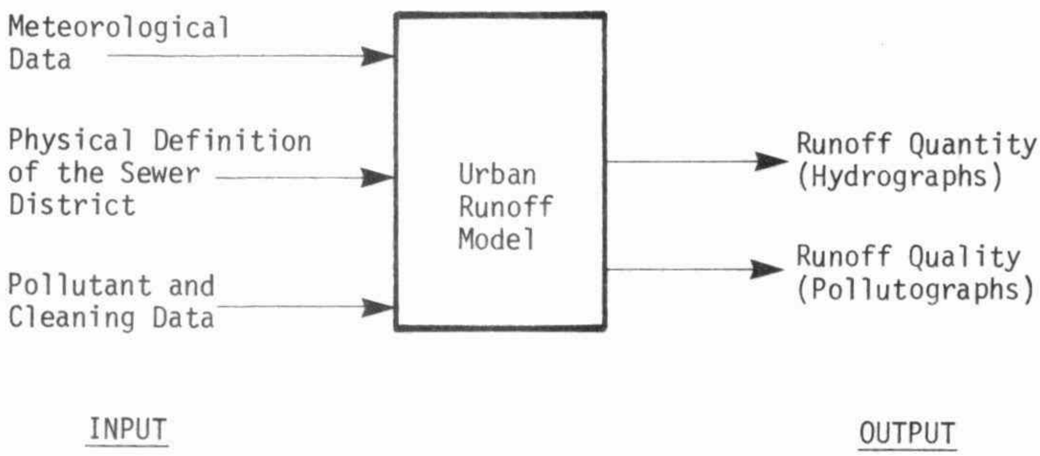
Comprehensive sampling of runoff from urban watersheds in Canada has only recently been instigated, and it will take years until data banks covering all specific conditions can be established. By that time, considerable investment could be made on the basis of very approximate methods, such as the rational method, or rules of thumb such as interception and treatment of 2-4 times the dry weather flows and the isolated upgrading of treatment plants.

Although relatively novel, the use of quality simulation models either for a first hand analysis or in conjunction with a more limited measurement programme is considered very helpful in answering the questions posed above. Quality simulation urban runoff models have already been used in a number of cities in the U.S., including Washington, San Francisco, Boston etc. [16,17,18]. Recently the U.S. Army Corps of Engineers' has recommended that STORM be used in all planning studies concerning the environmental effects of new urban developments [20]. STORM is at present being used in a study, conducted by APWA, for the determination of the impact of urban runoff and the potential for storage and treatment of runoff in a large number of American and Canadian municipalities. STORM was used in the assessment of the environmental impact of new developments in Pickering, Ontario [19]. Other practical applications of urban runoff quality modelling were carried out in Hamilton and Winnipeg. An appreciation of the problems of urban runoff pollution has been indicated in the recent Thames River Basin Report [21]. Storage ponds are also under consideration as part of a master drainage project, involving SWMM flow simulation, for the Township of Vaughan.

At present, the applications of new stormwater management techniques in Canada are mainly the result of the isolated efforts of individual municipalities, but a large scale implementation programme is at present under consideration. The present report forms one of the background studies in this programme.

1.3 TYPES OF MODELS

The broadest definition of the function of a model would be " the transformation of an input to an output", as indicated by the example given in the following diagram.



A good model is not only a predictive tool which generates additional information, it also gives a better insight into processes being studied and generates questions that may lead towards the reformulation of the initial problem.

Different urban runoff simulation models may be used at different stages of a project and consequently these operate at different levels of sophistication and have somewhat different objectives. Because of this, it is not sound to use a single model to investigate all facets of a problem.

(a) Planning models are used for an overall assessment of the urban runoff problem as well as for preliminary estimates of the cost effectiveness of different abatement procedures. Planning models may be used for the rating of alternative development schemes. Even if the results of quality simulations are not exact, they can indicate the most effective controls and the best areas for their location.

(b) Design/Analysis Models are oriented towards the detailed simulation of storm events. The models may be used for accurate predictions of flows and (eventually) concentrations at any point in the urban system and, in many cases, into the receiving waters. These simulations should illustrate the manner in which different abatement procedures or design options affect flow and quality at the critical locations. Data requirements for these more sophisticated models are usually more extensive than for the planning type of model.

(c) Operational models are used to produce actual control decisions during a storm event. Rainfall is entered from telemetered stations and the model is used to predict system responses a short time into the future.

In this report only (a) and (b) have been investigated.

Detailed design/analysis studies usually involve the simulation of a limited number of single isolated events, for which the antecendent conditions (e.g. soil moisture, pollutant accumulation etc.) have to be assumed. Usually, the limitations on the number of events modelled are those of computation cost and data requirements.

Planning studies for pollution abatement purposes and flood control require the use of a continuous simulation model. The results of such a model are usually presented in a form of a statistical interpretation of the system response over a long period. These models are generally founded on the same basic fundamental principals as the single event models. Some are high-speed screening models, which are necessarily fairly simple in order to minimize computer and data requirements. Screening models may eventually be coupled with optimization models in which one or more parameters (e.g. capital cost, instream oxygen, etc.) are optimized and a decision area is defined.

Most models used in urban runoff quantity simulation are deterministic, (i.e., follow the basic physical phenomena) while others are of the so-called "black-box" type, such as unit hydrograph models. Some quality models are of a correlation type and these models usually only operate successfully when used on the watershed for which they were developed. The deterministic models (such as SWMM and STORM) have a greater transferability, because of their more fundamental formulation. However, all models include parameters which originate from statistical studies or reflect some calibration or fitting to measurements. Before using a model the potential user should investigate past studies to ensure that the model is suitable for the intended application.

The most efficient model is not always the most sophisticated available. More sophisticated models require more voluminous data, which may not be available in the initial or planning stages of a study. As the framework of the study is more firmly established, and definite criteria for pollution and/or hydraulic control are agreed upon, the use of the more complex model will be necessary.

The review of models for potential use in Canadian conditions indicated that, in the category of "one event simulation models", the Storm Water Management Model (SWMM), developed by the U.S. Environmental Protection Agency was one of the most promising. It is the only currently available model integrating all aspects of runoff quantity and quality simulation, treatment and effects on the receiving water body. The SWMM is the result of many years of effort by several leading organizations including, the University of Florida in Gainesville, Water Resources Engineers (WRE) and Metcalf and Eddy. Since the first version of the model was published in 1971, it has been continuously improved and, at the present time, there are two updated versions of the model available. In the near future a new version of the SWMM will incorporate the sophisticated flow routing procedures developed originally by WRE for the simulation of surcharged conditions and complex flow behaviour. This version is required for more advanced design work involving pressurized flow conditions and may be used for design of relief sewers as well as new drainage studies.

The E.P.A. has an intensive programme of technology transfer seminars and publications which ensure that the model has, and will continue to receive, widespread attention and frequent testing and improvement. SWMM may be used for planning and in some cases for design/analysis.

In the category of Planning models, a more approximate Long Term Simulation Model is also required. STORM was developed by Water Resources Engineers Inc. for the U.S. Army Corps of Engineers' for this purpose. This is a simple, high speed model, which incorporates continuous quality modelling routines. Because of the Corps' recommendation for the use of STORM in planning studies, the model is certain to receive widespread use.

Brief descriptions of these models are given in the succeeding sections. A comparison of different features of SWMM, STORM and many other models is summarized in Table 1.1 [22].

1.4 DESCRIPTION OF THE SWMM

SWMM is a comprehensive mathematical simulation model which requires a high speed digital computer for the simulation of storm events. The inputs to the model include a rainfall intensity distribution in time (hyetograph) and data describing the idealized catchment, transport and receiving water systems. The principal objective of the model is the complete characterization of the temporal and spatial effects of a storm [23]. To achieve this, the flow and associated pollutional aspects are represented as continuous curves referred to as hydrographs and pollutographs.

The urban drainage system may be illustrated as a series of inter-related subsystems [24], Figure 1-1. The SWMM allows the effect of each of these subsystems on the stormwaters to be simulated. A simplified overview of the model, shown in Figure 1-2, indicates the major processes performed within the model blocks. These are briefly discussed in this section while a complete description may be found in Volume 3 of this report, the User's Manual.

RUNOFF BLOCK - The rainfall hyetograph is distributed evenly over the subcatchment within each time interval (typically 5 minutes). The drainage area is characterized by its size, degree of imperviousness, slope and several factors describing the pollutant accumulation over the area. The available depression storage and infiltration potential are satisfied before runoff occurs. The overland flow is then considered using the kinematic wave formula based on Manning's equation and continuity at each time interval. The overland flow may be routed through small pipes and gutters in its travel to the inlet manhole. The rate of overland flow determines the amount of the available surface pollutants incorporated in the flow. Thus, at the inlet, a temporal description of the flow and the pollutant mass washoff is available. These hydrographs and pollutographs, which are the output from the RUNOFF block, form the input to the TRANSPORT block.

<u>TRANSPORT BLOCK</u> - This is used to represent the physical works that convey surface runoff hydrographs and pollutographs from the contributing subcatchments to the point of discharge. This may be a storage treatment facility, or a receiving water. The inlet flows are modified by combination with flows from other drainage areas and by any dry weather flow in the system. The flows and pollutant discharge rates are attentuated during their passage through the pipe network to an extent depending on the capacity of the system and the function of in-system storage.

The solution procedure follows a modified kinematic wave approach in which disturbances are allowed to propagate only in the downstream direction. The resultant flow velocity in each element controls the deposition or scour of solids. Flows may be diverted at various points in the system, either to overflow locations or to treatment facilities.

<u>STORAGE/TREATMENT BLOCK</u> - The output from the TRANSPORT block, or some portion of this, such as 3 x DWF, may be routed to the storage units. These are specified by simple regular geometries with various alternative inlet and outlet controls. Pollutants are considered to settle out in the storage facility, while additional improvements in quality may be modelled in the variety of optional treatment units. The resulting attenuated hydrographs and pollutographs can form the input to the RECEIV block.

<u>RECEIV BLOCK</u> - Any receiving water may be modelled in this block. The geometry of the actual system is described using a network of nodes and channels or a simple river system may be considered as a series of channels. The output from either the TRANSPORT or STORAGE/TREATMENT blocks, or both, may be used as the input to different nodes in the receiving water and the

subsequent responses investigated. The equations of continuity, motion and conservation of mass are solved in order to establish the variations in water level and pollutant concentration at nodes of interest over several days. The effect of different treatment policies may be compared in a series of runs using only the quality subroutine of this block.

1.5 DESCRIPTION OF STORM

Figure 1-3 shows a conceptual view of the urban system considered in STORM. The model operates with an hourly record of precipitation and temperature, which may extend over a large number of years. The catchment is described in terms of land use and imperviousness. Different pollutant accumulation rates are associated with the various land uses. The rainfall or snowmelt in excess of the available depression storage is transformed directly to runoff at the outlet from the catchment. Flow and quality routing is not considered, the pollutograph being directly related to the runoff rate in any hour. A treatment rate for this runoff may be supplied. Flows in excess of this rate may be stored or considered as direct overflow. The water balance between storms is very simply considered as the recovery of depression storage by evapotranspiration, while the surface pollutant accumulation is modified by street cleaning at a specified frequency. For a given rainfall/snowmelt record, the quantity, quality and number of overflows will vary as the treatment rate, storage capacity and land use is changed.

The model is considered in more detail in Chapter 11.

	CATCHMENT HYDROLOGY							SEWER HYDRAULICS							
	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	SEVERAL RAINGAGES	SNOWMELT	RUNOFF FROM IMPERVIOUS AREAS	RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS	OPEN CHANNEL NETWORK	UPSTREAM & DOWN-STREAMFLOW CONTROL	SURCHARGING AND PRESSURE FLOW	DIVERSION STRUCTURES	PUMPING STATIONS	STORAGE	COMPUTES STAGE	COMPUTES VELOCITIES
BATTELLE NORTHWEST	●	●	●		●	●		●			●		●	●	●
BRITISH ROAD RESEARCH LABORATORY	●	●	●		●			●							
CHICAGO FLOW SIMULATION	●	●	●	●	●	●	●	●					●	●	
CHICAGO HYDRO-GRAPH METHOD	●	●	●		●	●	●	●							
COLORADO STATE UNIVERSITY	●	●			●			●	●					●	●
CORPS OF ENGINEERS				●	●	●	●				●		●		
DORSCH CONSULT	●	●	●		●	●		●	●		●		●	●	●
ENVIRONMENTAL PROTECTION AGENCY	●	●	●		●	●		●	●		●	●	●	●	●
HYDROCOMP	●	●	●	●	●	●	●	●	●	●	●		●	●	●
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	●	●			●	●		●						●	●
MINNEAPOLIS-ST. PAUL	●	●	●		●	●		●			●				
SEATTLE METRO									●		●	●	●		
SOGREAH	●	●			●	●		●	●	●	●	●	●	●	●
UNIVERSITY OF CINCINNATI	●	●			●	●		●						●	●
UNIVERSITY OF ILLINOIS	●	●			●	●		●	●				●	●	●
UNIVERSITY OF MASSACHUSETTS	●	●			●		●		●					●	●
WATER RESOURCES ENGINEERS	●	●	●		●	●		●	●	●	●	●		●	●
WILSEY AND HAM	●				●	●		●							

TABLE 1.1

MODEL CHARACTERISTIC [22]

	WASTEWATER QUALITY									MISCELLANEOUS				
	DRY-WEATHER QUALITY	STORMWATER QUALITY	QUALITY ROUTING	SEDIMENTATION AND SCOUR	QUALITY REACTIONS	TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION	CONTINUOUS SIMULATION	CAN CHOOSE TIME INTERVAL	DESIGN COMPUTATIONS	REAL-TIME CONTROL	COMPUTER PROGRAM AVAILABLE
BATTELLE NORTHWEST	●	●	●			●		●	●		●	●	●	●
BRITISH ROAD RESEARCH LABORATORY								●			●			●
CHICAGO FLOW SIMULATION								●		●	●			●
CHICAGO HYDRO-GRAPH METHOD								●			●	●		●
COLORADO STATE UNIVERSITY														●
CORPS OF ENGINEERS		●			●		●			●				●
DORSCH CONSULT								●	●					
ENVIRONMENTAL PROTECTION AGENCY	●	●	●	●	●	●		●	●		●			●
HYDROCOMP	●	●	●		●		●	●	●	●	●	●		
MASSACHUSETTS INSTITUTE OF TECHNOLOGY								●			●			●
MINNEAPOLIS-ST. PAUL											●			●
SEATTLE METRO											●		●	●
SOGREAH								●			●			
UNIVERSITY OF CINCINNATI								●			●	●		●
UNIVERSITY OF ILLINOIS											●			●
UNIVERSITY OF MASSACHUSETTS														●
WATER RESOURCES ENGINEERS	●	●	●		●			●	●		●			
WILSEY AND HAM											●	●		

TABLE 1.1 (cont'd)

REFERENCE - CHAPTER 1

1. Koplyay, T.M., "Urban Runoff Problems in Canada" a paper presented at the American Public Works Association Congress in New Orleans, September 1975.

2. "Storm Water Management Model" Volume 1, Final Report prepared by Metcalf and Eddy Inc. et al for the U.S. EPA, contract No. 14-12-501, July 1971.

3. "Urban Storm Water Runoff, STORM", A Generalised Computer Program, H.E.C. Corps of Engineers, U.S. Army, Davis, California, January 1975.

4. Watkins L.H., "The Design of Urban Sewer Systems", Road Research Technical Paper No. 55, Department of Scientific and Industrial Res. London, H.M. Stationary Office, 1962.

5. Kiefer, C.J., Harrison J.P. and Hixson, T., Unpublished preliminary report "Chicago Hydrograph Method", July 1970.

6. Klym, H., Koniger, W., Mevius, F., Vogel, G., "Urban Hydrologic Process - Computer Simulation" Dorsch Consult, Munich. (Via Dorsch Consult, 45 Richmond West, Toronto, Ontario.)

7. Brandstetter, A.B., "Assessment of Mathematical Models for Storm and Combined Sewer Management", an Unpublished preliminary report. Battelle Pacific Northwest Laboratories, Richland, Washington 99352.

8. Kidd, C.H.R., "The Development of an Urban Runoff Model" Unpublished MSc Thesis Department of Civil Engineering, Queen's University at Kingston, 1972.

9. U.S. Environmental Protection Agency, "Storm and Combined Sewer Technology Program", Municipal Environmental Res. Lab., Cincinnati, Ohio, Edison New Jersey 08817.

10. "San Francisco Storm Water Model - User's Manual and Program Documentation" prepared for the City and County of San Francisco by Water Resources Engineers, available via Water Resources Engineers Inc., Walnut Creek, California 94596.

11. Heeps, D.P., Mein, R.G., "An Independent Evaluation of Three Urban Storm Water Models", Monash University Civil Engineering, Res. Report No. 4, 1973.

12. Linsley, R.K., "A Critical Review of Currently Available Hydrologic Models for Analysis of Urban Storm Water Runoff", Office of Water Resources Research, Washington, D.C., 1971.

13. "Review of Canadian Storm Sewer Design Practice and Comparison of Urban Hydrologic Models", prepared by James F. Maclaren Ltd. for Canada - U.S. Agreement Task 3/4 Project 4, October 1975, Project No. 74-8-31, obtainable from Training and Technology Transfer Division (Water), Environmental Protection Service, Environment Canada, Ottawa, Ontario, KlA OH3.

14. James F. MacLaren Ltd: Edmonton, Mississauga, Vaughan, Toronto International Airport, Proctor & Redfern Ltd.: Toronto International Airport, Hamilton, Gore and Storrie Ltd.: Ottawa.

15. Clarke, W.G., et al "Hydrograph Methods in Relief Sewer Design - A Case Study" an Unpublished paper presented at the SWMM user's Meeting, University of Florida, Gainsville, Florida, February 1975.

16. Buckingham, P.L., Shih, C.S., Ryan, J.G., Lee, J.A., Kane, J.K., "Combined Sewer Overflow Abatement Alternatives Washington, D.C.", Water Pollution Control Research Series 11024 EXF 08/70, Environmental Protection Agency, 1970.

17. "The Use of Stormwater Simulation Models in Developing a Coordinated Waste Water Management Plan for the City of San Francisco", A Summary Report prepared by Water Resources Engineers for the Division of Sanitary Engineering, Department of Public Works, City and County of San Francisco, September 1974.

18. Pisano, W.C., "Cost Effective Approach for Combined and Storm Sewer Clean-Up", obtainable via Environmental Planning, Energy and Environmental Analysis, Inc. Boston, Massachusetts.

19. James F. MacLaren Ltd., "The Hydrological and Environmental Impacts of Development Alternatives on the North Pickering Community" an unpublished report for the Plantown Consortium, April 1975.

20. Engineer Technical Letter No. 1110-2-515, 28 February, 1974 "Urban Studies Program, Management of Urban Stormwater Runoff" Department of the Army, Office of the Chief of Engineers, Washington D.C. 20314.

21. "Water Management Study - Thames River Basin" a report prepared by the Ministry of the Environment and the Ministry of Natural Resources, Ontario, 1975.

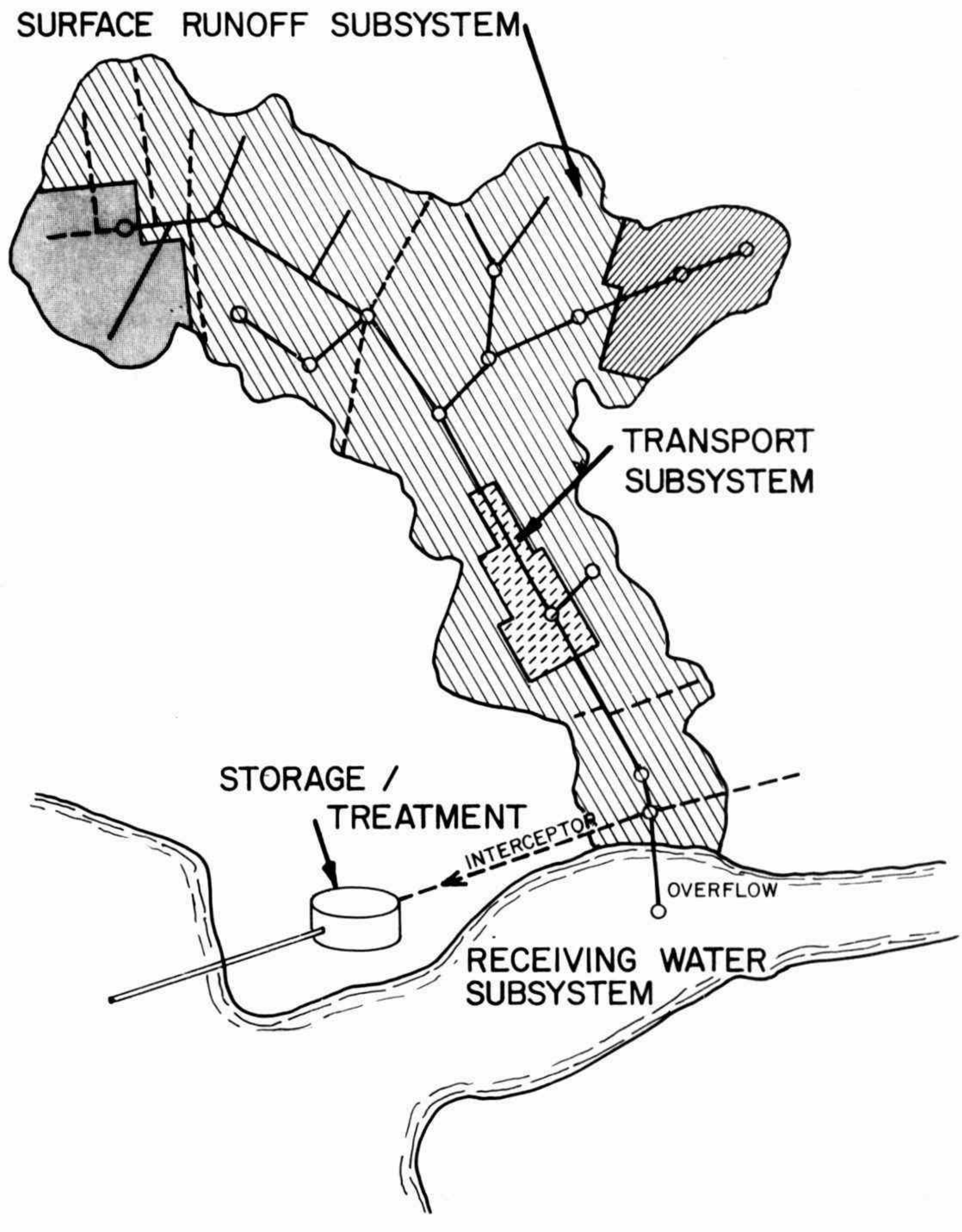
22. Torno, H.C., "Storm Water Management Models", Urban Runoff Quantity and Quality, Proceedings of a research conf., Franklin Pierce College, Rindge, New Hampshire, August 1974, published by ASCE.

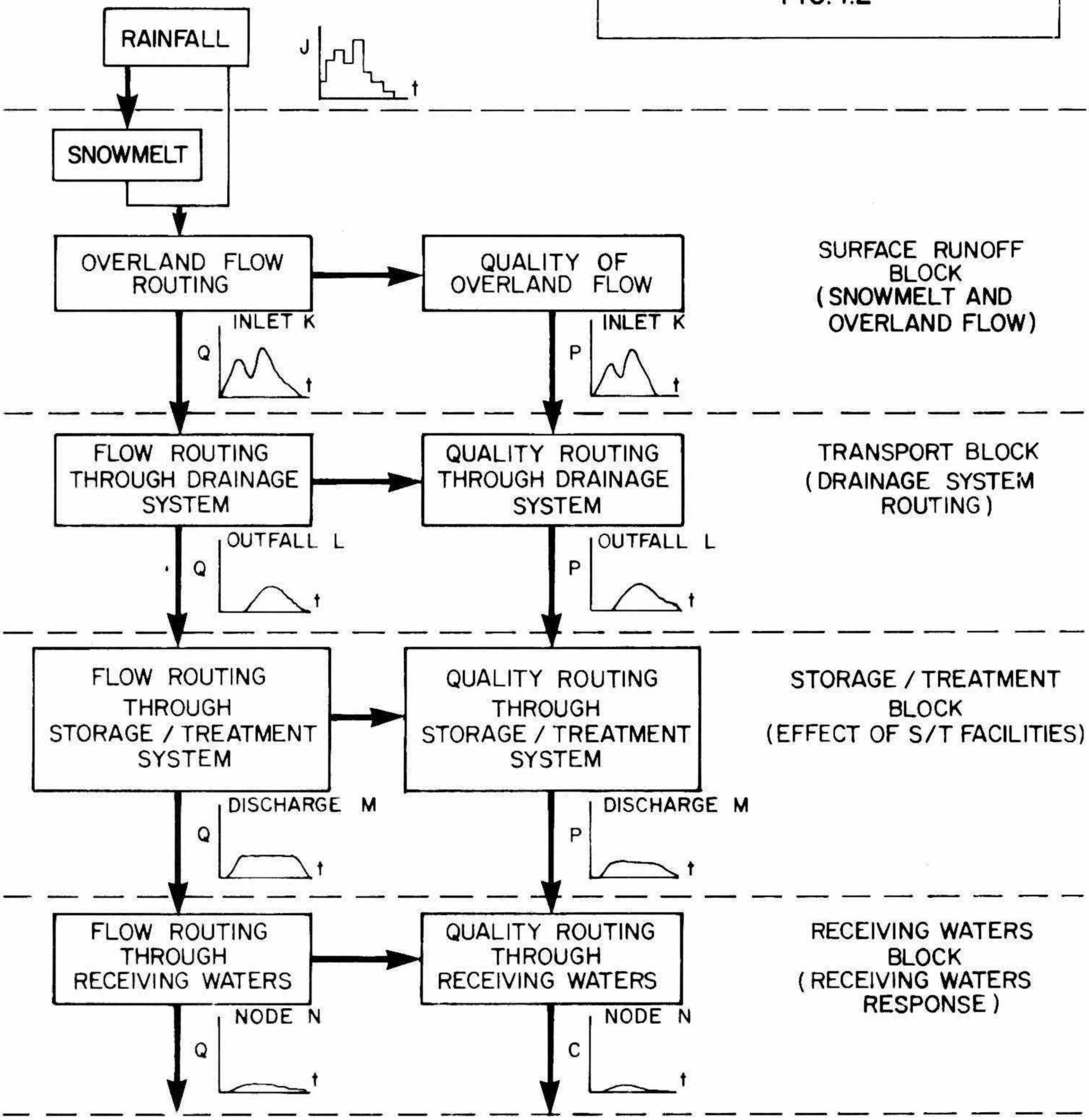
23. Huber, W.C., Heaney, J.P., "Introduction to the EPA Storm Water Management Model" a paper presented at the 1975 Short Course, Applications of Stormwater Management Models, University of Massachusetts.

24. "Management of Urban Storm Runoff", ASCE Urban Water Resources Research Program. Technical Memorandum No. 24, WRE and HEC, US Corps of Engineers, May 1974.

THE URBAN DRAINAGE SYSTEM

FIG. 1.1



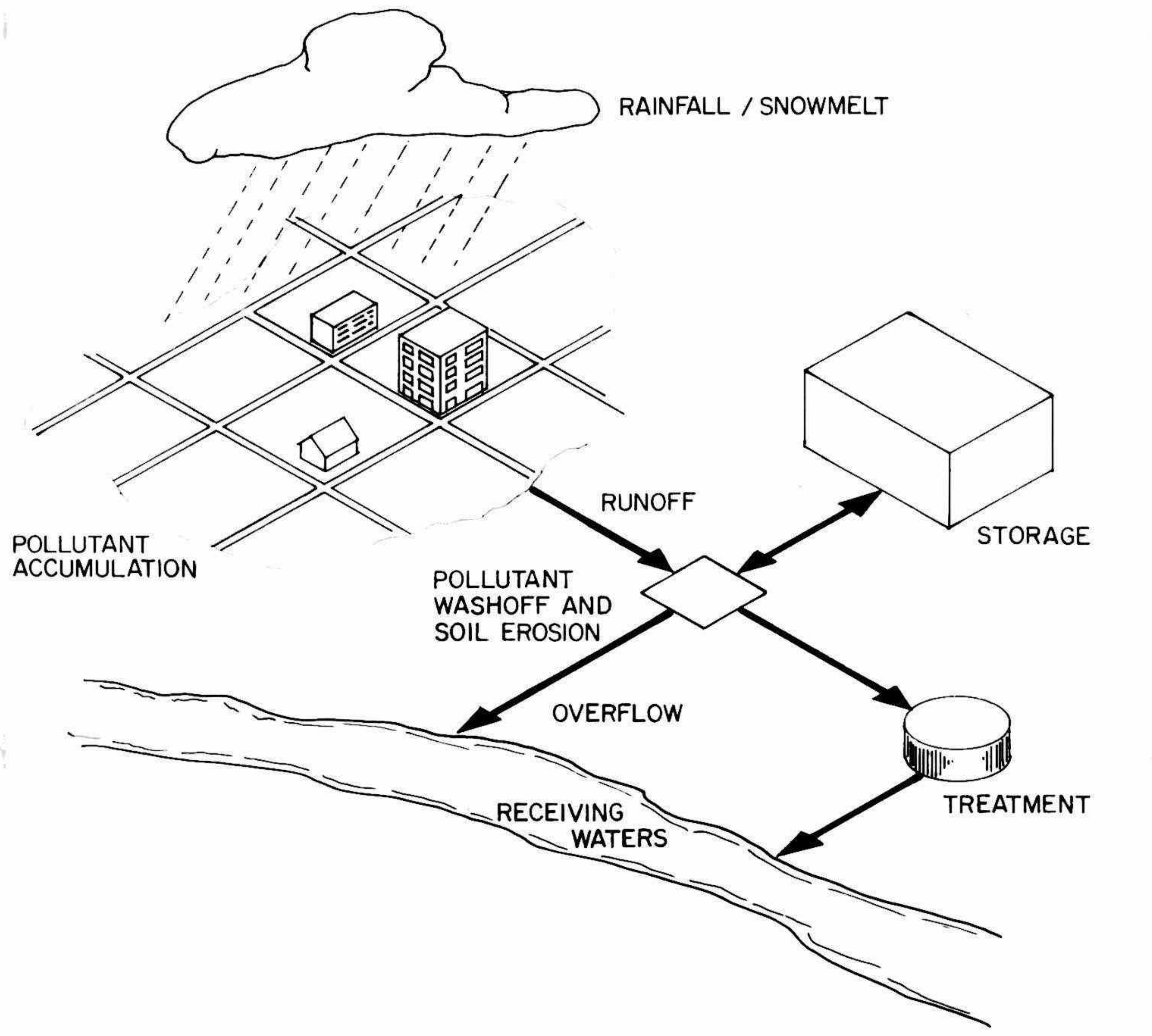


OVERVIEW OF THE STORM WATER MANAGEMENT MODEL

FIG. 1.2

CONCEPTUALISED VIEW OF URBAN SYSTEM IN STORM

FIG. 1.3



SCOPE AND ORGANIZATION OF STUDY

CHAPTER 2

CHAPTER 2
SCOPE AND ORGANIZATION OF STUDY

2.1 BACKGROUND

Early in 1974, the situation with regard to stormwater management modelling in Canada was as follows:

- there were many Urban Runoff models available, each with a different degree of sophistication, and suitability for particular tasks.
- the Urban Runoff Study carried out for the Canadian Centre for Inland Waters [1], concluded that the EPA SWMM held most promise, primarily due to its comprehensive consideration of the urban system, accuracy, excellent documentation and widespread acceptance.
- Only the RUNOFF block of the SWMM had been tested with Canadian data. Comparison of RUNOFF simulations with measurements were limited to watersheds less than 100 acres in size.
- there was an obvious need to verify the SWMM against measurements for larger watersheds and for a broader range of meteorologic conditions.
- a submodel for the simulation of snowmelt quantity and quality was not yet available, but would certainly be needed by Canadian planners and engineers.
- perhaps due to the large number of U.S. researchers working on the SWMM and other Urban Runoff models, there were always several versions of each available. Not all of these were fully operational. It was therefore

necessary to ensure that all SWMM submodels were operational and compatible.

- to ensure that the models respond logically to reasonable variations of input parameters, systematic sensitivity analyses would be required.

- the degree of sophistication of model input data had not been defined in relation to the level of accuracy required in the model output.

- in general, the more sophisticated the model, the greater the cost of data preparation and data processing. The quality of results do not always increase with increasing sophistication. Further investigation appeared to be needed in the areas outlined below:

  - the SWMM TRANSPORT routine should be compared with the more sophisticated routines used in the Dorsch HVM (Hydrograph Volume Method) and WRE (Water Resources Engineers) models, particularly with a view to detailed design applications.

  - the Receiving Water Body routine in the SWMM is rather more simple than some comparable water quality models. An assessment of its limitations and its applicability to planning studies would be useful.

  - in the SWMM, pollutants are routed through the sewer system and pollutant decay is modelled in transit. In most practical cases, the available data would not be sufficient either in quantity or quality to justify such precise simulation. Simplified routines should therefore be investigated.

- There was a need for recommendations describing the appropriate selection of various optional routines on the SWMM in specific circumstances.

- Cost routines in the SWMM were based on U.S. cost figures for equipment and services. These should be modified for Canadian conditions.

- There was a requirement for a programme of testing and verification of the STORM model, similar to that for the SWMM.

- Because of the large amount of meteorological data required, especially for long term simulation, a program to extract and manage the necessary data was required. Such a program should be capable of extracting information from the Atmospheric Environment Service data banks and processing these in such a format as to be suitable for direct input to the long term simulation model STORM and the SWMM.

2.2 TERMS OF REFERENCE

(1) Review and modification of the U.S. EPA Storm Water Management Model (SWMM) for Canadian conditions.

The Contractor shall review the latest version of the EPA SWMM and modify some subsystems of the model as discussed in the following paragraphs. The modified model has to be fully compatible with the SWMM version maintained at the University of Florida, with respect to the model structure, interaction between the main program and subroutines, and input data requirements.

(a) Executive Block

The Executive Block will be modified only to the extent required by addition of new subsystems, e.g., the data subsystem.

(b) Runoff Block

Runoff Block generates surface runoff based on an arbitrary rainfall hyetograph, antecedent conditions, land use and topography. This block is built of seven subroutines which act as follows:

RUNOFF	-	Liaison with EXECUTIVE BLOCK
HYDRO	-	Co-ordinates hydrograph computation using RHYDRO, WSHED, GUTTER
RHYDRO	-	Reads and prepares input data
WSHED	-	Computes overland flow
GUTTER	-	Computes flow in pipes/gutters
HCURVE	-	Prepares computed hydrographs for plotting
SFQUAL	-	Calculates pollutants washed off the land

Only minor modifications of this Block will be executed, namely, modifications resulting from study of the limits of a discretization of the watershed elements (equivalent blocks) and from changes to the surface water quality subroutine (SFQUAL).

i) Study of Equivalent Catchments

The level of detail necessary to adequately represent watershed elements shall be investigated. The objective of this aspect of the work will be to illustrate the effect of reducing the number of subcatchments on the accuracy of runoff simulation. While this analysis will be empirical, confined to testing on one or two watersheds, an effort will be made to define the considerations which would more generally apply to selecting the detail necessary to achieve the desired accuracy.

Necessary modifications will be made to RUNOFF, RHYDRO, WSHED and, perhaps HYDRO and GUTTER.

ii) Modification of the Surface Water Quality Subroutine (SFQUAL)

The surface water quality subroutine was developed using, in some cases, limited and localized data. The Contractor shall review

the entire subroutine and shall include new estimates of co-efficients and model parameters, based on the results of ongoing Canadian as well as other studies. In particular, the parameters used to compute the following items shall be reviewed:

Pollutants accumulated on ground surface (prior to the storm)
Availability factor for pollutants on ground (during the storm)
Estimates of dust-dirt fall
BOD in catchbasins at start of storm and removal rate
Coliforms in surface runoff

The surface water quality shall be described by water quality parameters such as: BOD, COD, DO, Coliforms, Phosphorus, Nitrogen and suspended solids. Parameters may be added to, or deleted from, the above list pending Contractor's review of the current studies and discussion with the Scientific Authority.

The Contractor shall identify the basic types of behaviour of the above quality parameters and processes such as those involving balance or decay with time. The model shall have flexibility to enable it to be operated using any or none of these parameters within these groups for specific watershed studies.

(c) <u>Transport Block</u>

The Transport Block routes flows in sewers, routes water quality, estimates dry weather flow, infiltration, and treats in-system storage. The Block is constructed as follows:

TRANS	-	Liaison with EXECUTIVE BLOCK
TSTRDT	-	Reads and prepares input data
SLOP	-	Sequences sewer elements
FIRST	-	Computes constants and flow parameters in each flow element
INFIL	-	Estimates and allocates infiltration
FILTH	-	Computes dry weather flow quality and quantity

DWLOAD	-	Computes initial sediment in sewers
INITIAL	-	Initializes flows, areas, pollutant concentrations based on DWF infiltration
QUAL	-	Routes pollutants through sewer
ROUTE	-	Routes flow through sewer
PRINT	-	Prints hydrographs and pollutographs
TSTORG	-	Computes in-sewer flow storage

The Contractor shall concentrate his effort on the following three areas:

(i) Flow routing model (ROUTE)
(ii) Quality routing model (QUAL and DWLOAD)
(iii) Infiltration model (INFIL)

i) Flow Routing Model (ROUTE)

This subroutine shall be reviewed and its performance compared to other routing models. The range of applicability of the present EPA subroutine will be defined as well as its limitations, possible improvements and increased costs associated with more sophisticated procedures. The Contractor shall recommend and implement the most appropriate (non-proprietory) routing scheme or schemes and will present the reasons for preference. The scope of this work shall be further reviewed and discussed with the Scientific Authority.

In selecting the routing scheme, primary importance shall be placed on the model capabilities in handling both backwater and surcharge conditions, and the following control elements:

Manholes
Pump Lift Stations
Flow Dividers (linear and non-linear)
In-line Storage Units
Weirs

The routing model shall be capable of executing the above calculations for conduits of various shapes frequently used in engineering practice.

ii) Quality Routing Model (QUAL and DWLOAD)

The Contractor shall review the quality routing model and apply a sensitivity analysis to the model. The following model parameters will be used in the sensitivity analysis:

Oxygen utilization rate
Reaeration rate
Saturation value
Specific gravity of sewer sediments

The above analysis may reveal if a simplification of the model is feasible and, if so, the appropriate simplification will be implemented.

iii) Infiltration Model (INFIL)

The Contractor shall review and test the infiltration model by comparing its predictions to real data. If appropriate, the model will be simplified.

(d) <u>Storage Block</u>

The Storage Block incorporates a number of schemes for control and abatement of pollution due to discharges and overflows from sewer networks. The main functions of the block are as follows:

(i) Routing of sewage flows through an off-systems storage
(ii) Treatment of sewage
(iii) Cost estimation for the schemes considered under Items (i) and (ii)

The Contractor shall concentrate his effort on the treatment model and cost effectiveness model.

i) Treatment Model

The heart of the Treatment Model is subroutine TREAT. It computes the movements and removals of water and pollutants on a process-by-process basis, every time step. It, in turn, makes use of the following subroutines:

STRAGE	-	Models storage facilities
BYPASS &		
TRLINK	-	Link up treatment processes
SEDIM	-	Models sedimentation
KILL	-	Models coliform reduction
HIGHRF	-	Models high rate filter operation

The treatment process presently handled by the model is:

Bar racks
Fine Screens
Sedimentation
Dissolved air flotation
Microstrainers
High rate filters
Effluent screens
Chlorination

The Contractor shall update each of the process models, taking into account variations in treatment efficiency. Process models shall be developed for processes not included in the present model version but which are currently being investigated, such as the disposal of effluent. This will require modifications of BYPASS, TRLINK, TREAT and perhaps the EXECUTIVE BLOCK liaison subroutine STORAG which handles the Treatment Model.

The scope of this development work shall be subject to review by the Scientific Authority.

ii) Cost Effectiveness Models

Two cost models are used in the SWMM: TSTCST in the TRANSPORT BLOCK, and TRCOST in the STORAGE BLOCK. Together they make up the COST EFFECTIVENESS MODEL.

TRCOST	-	Computes estimated costs for provision of storage and treatment facilities specified and operating and maintenance costs during storm event modelled
TSTCST	-	Computes capital and operating costs of the designated in-line storage units

Both subroutines employ construction cost indexes, assumed land cost, assumed volume/cost relationships, etc., arriving at estimates.

The Contractor shall update and relate to Canadian conditions all assumed costs and cost formulas built into the model.

(e) Receiving Water Block

The Receiving Water Model consists of two major submodels: a flow model (SWFLOW) and a quality model (SWQUAL). Each submodel employs a number of smaller subroutines to effect the specific hydraulic and quality calculations.

The existing SWMM receiving water block will be included as part of the work and the advantages and disadvantage of acquiring or developing, and implementing other models, will be described.

(2) Selection and modification of a submodel for the simulation of quantity and quality of runoff resulting from snowmelt.

Under Canadian conditions, snowmelt events can be accompanied by intense rainstorms in spring or occasionally, in late fall and may contribute to flooding in urban areas, even in the case of small basins. In addition, in the urban areas of Canada, water

quality during the winter may be significantly affected by the use of various chemicals on the streets for snowmelting purposes and, possibly to some extent, by increased air pollution due to heating, inefficient use of cars, etc.

In view of this, the Contractor shall pay particular attention to the problem of modelling snowmelt with superimposed rainstorms and to the related problems of water quality.

Snowmelt water quantity modelling shall consist of modifying existing snowmelt models to apply to urban conditions, including effects of chemical additives and snow removal. The data requirements of the snowmelt model selected shall match the capabilities of the Data Analysis submodel described below.

Input data required may consist generally of precipitation, temperature, solar radiation, wind, humidity and cloud cover, and shall be obtained, using the data bank maintained by the Meteorological Branch. For those cases in which not all the required data are available, the Contractor shall indicate how these data could be estimated.

The effect of a rainstorm on the snowmelt conditions shall also be modelled, using existing approaches.

Insofar as practical, the estimated accuracy of the snowmelt simulation will be consistent with the capability of the SWMM, developed under this contract, to predict runoff generating from rainstorms.

The quality of snowmelt water shall be modelled, accounting for street salting and pollutants retention by snow. The list of water quality parameters to be considered shall be determined by the Contractor on the basis of results of current studies and discussions with the Scientific Authority.

The snowmelt model shall be properly interfaced with the rest of the SWMM.

(3) Development of a Data Analysis Submodel

An important aspect of the proposed study is to ensure that the model will be compatible with existing data banks. From the point of view of real storm analyses or continuous simulation, the most important such data bank is the Climatological Bank of the Canadian Atmospheric Environment Service. Hourly-sampled climatological data, describing rain events and phenomena influencing snowmelt, are available on magnetic tape or punched cards from the Atmospheric Environment Service. Subroutines shall be written to create and manage a data/base for the detailed and high speed models from such existing data banks. These shall include the capability of distributing rainfall intensity data to different basin subcatchments. Recommendations for future urban rain gauge networks shall be made.

The capabilities of the Data Analysis submodel shall not be restricted to handling hourly-sampled data since, in future surface watershed studies, data may be available for shorter sample intervals.

The Contractor shall provide detailed documentation for all the above data analysis subroutines and shall interface the data submodel with the rest of the model. Formats of data made available through the data submodel shall be fully compatible with the data formats required for the EPA SWMM.

The Contractor's effort will be co-ordinated with the EPA ongoing R&D project to establish urban hydrology data base.

(4) Selection and modification of a high-speed model for simulation of frequencies of discharges and overflows.

The Contractor shall select and modify, where necessary, a high-speed model for the long-term simulation of runoff events. The

model shall have the capability of simulating the runoff quantity and quality, overflows, and effects of storage and treatment (capacity) on the aforementioned phenomena.

The model developed will be checked against the detailed model (SWMM) as well as against real data (e.g., southeast Toronto Watershed).

The Contractor shall provide interfacing between the high-speed model and data files and between the high-speed model and the detailed model (SWMM).

(5) Additional Considerations

(a) Implementation on Computer

The computer programs developed and modified as part of this study shall be written in FORTRAN. The total SWMM model, including integrated submodels selected or modified as part of this work, shall be tested, verified, and submitted, using a computer system compatible with the latest verison of the SWMM submodels.

The Scientific Authority is desirous that the SWMM system and extensions incorporated as part of this work be suitable for execution on competitive large-scale computers of manufacturers other than the one used in this study. To this end, the Contractor shall submit to the Scientific Authority, three months before the end of this study, suitable terms of reference, including cost estimates for the conversion, testing, and verification of the SWMM system and extensions as incorporated in this study.

Insofar as pracitcal, the Contractor will consider the idea of program implementation on large-scale computers, such as IBM, CDC, UNIVAC, when developing new programs during this study.

The cost of using computers for watershed modelling includes both the cost of machine time and the cost of engineering time. Inter-

active time-sharing via low-speed terminal will increase computer cost but may provide significant savings in engineering cost. The Contractor shall review and comment on the relative merits of batch processing with normal turn-around in the Toronto area and interactive time-sharing, considering the various phases of urban watershed modelling.

Guidelines shall be developed to allow users to estimate cpu time on at least one hardware configuration before running the SWMM.

All documentation of the new submodels shall be written using the format of the New User's Manual being compiled by the University of Florida. A master tape of the source programs, along with an appropriate Operator's Manual, shall be included with the final report.

(b) Verification

It is expected that each subsystem will be verified by comparing its performance to either existing Canadian data or to data available elsewhere, pending the approval by the Sciencific Authority.

Where no appropriate verification data are available, a field program will be designed around the needs of this investigation and will be the subject of a separate terms of reference, requiring subsequent proposals. The Contractor shall identify such requirements in general terms within 30 days and on a continuing basis throughout the study.

The Contractor shall make maximum use of the pertinent field data available in Canada or elsewhere. The Contractor's efforts shall be co-ordinated with other Canadian research activities (mostly research sponsored by DOE, MOE and NRC and with U.S. Environmental Protection Agency Research Program.

(c) SWMM Model Parameters - Default Values

There are numerous default values built into the SWMM to allow simulation to continue when the value of a model parameter is not known. These default values were often based on very limited data during the development of SWMM. The Contractor shall review the default values and, where necessary, shall update them and relate them to Canadian conditions and practices.

2.3 STRUCTURE OF REPORT

The report is divided into four volumes, as follows:

Volume 1 - Final Report
Volume 2 - Technical Appendices
Volume 3 - User's Manual
Volume 4 - Program Listings

Volume 1 gives a description of test applications, sensitivity analyses for existing SWMM routines, suggested modifications and simplifications, and a description of new programs. A Technical reader will no doubt wish to read all of the volume. A general reader should be able to obtain a good appreciation of the study by reading only the Summary, Basic Concepts (Chapter 1) and the Conclusions and Recommendations (Chapter 13). The Final Report is, for convenience, divided into five parts:

I Introduction
II Application of the SWMM on Canadian Watersheds
III Modifications to the SWMM and Auxilliary Routines
IV Interfacing the SWMM with Continuous Simulation Models
V General Conclusions and Recommendations

Volume 2 describes previous research and reviews critically the state of the art for the benefit of more technically oriented readers. The 7 appendices are as follows:

1 Comparative Analysis of Routing Models
2 Comparative Analysis of Water Quality Models

3 Literature Survey - Urban Runoff Quality
4 Literature Survey - Snowmelt Quantity
5 Literature Survey - Snowmelt Quality
6 Data Collection - Study Areas
7 Treatment Processes

aspects of the study for those who will be directly involved in simulations. This manual includes the EPA manual for SWMM, modified to include some of the recommendations that emanated from this study. It also describes how to use STORM and the other programs developed.

Volume 4 gives program listings for SWMM and STORM on tape discs and for DAP and GSQM on printed listings.

REFERENCE - CHAPTER 2

1. James F. MacLaren Ltd., "Review of Canadian Design Practice and Comparison of Urban Runoff", Canada-U.S. Agreement Task 3/4 Project 4, October 1975, Project No. 74-8-31, Obtainable from Training and Technology Transfer Division (Water), Environmental Protection Service, Environment Canada, Ontario, K1A 0H3.

ASSESSMENT AND SELECTION OF STUDY AREAS

CHAPTER 3

CHAPTER 3
ASSESSMENT & SELECTION OF STUDY AREAS

3.1 GENERAL

Since the publication of a preliminary appraisal of stormwater overflows by the U.S. Department of Health, Education and Welfare in 1964 [1], a considerable number of monitoring and sampling programmes have been instituted in the U.S. and Canada. Initially these programmes were oriented towards overall characterization of pollutant discharges, annual averages, and regional variations, rather than towards precise definition of the complex inter-relationships determining urban runoff flow and quality from single events. However, the recent advances in computer modelling of the Urban Runoff process have created the need for precise, short interval stormwater quantity and quality data for single events, for the calibration, validation and refinement of these Urban Runoff Models. Much of the existing body of stormwater data is unsuitable for modelling purposes, and researchers have been forced to rely on limited data, or data from several sources. For example, Brandstetter [2] attempted to select three catchments with reliable rainfall, runoff and quality data on which to base rigorous comparisons between various models. Because of the limited availability of adequate data, the study was restricted to only two areas; Oakdale, a small 13-acre catchment with no quality data; and Bloody Run, a large 2380-acre catchment, the data from which appeared somewhat unreliable.

During the course of this present study, a number of promising collection programmes have been instituted to supplement this scarcity of reliable data and are currently underway. The University of Florida is conducting a programme for collection, classification and dissemination of all current data in an effort to provide model users and developers with a comprehensive data base. An interim report [3] identifies 42 potential data sources. However only a few of these are at present available, and these are mainly the older published data extensively using during the SWMM development. A review of the more promising of these potential sites is contained in Volume 2 of this report.

In the present study efforts were directed firstly to a review of available U.S. data, and to a subsequent investigation of Canadian studies which might be potential sources of modelling data. This chapter discusses the data requirements for urban runoff modelling purposes, and assesses the Canadian data which were used in this study.

3.2 DATA REQUIREMENTS FOR A STUDY WATERSHED

The SWMM requires an extensive amount of data to perform a detailed single event simulation. These data may be grouped into three categories:

(a) Meteorological data describing the primary input to the modelling system; short interval precipitation data, and temperature data if snowmelt is being modelled.

(b) Data describing the physical system; the nature and characteristics of the runoff producing area and its associated sewer system. These include the imperviousness ratio of the watershed, ground slopes, sewer system geometry, hydraulic roughness and other specific information.

(c) Data for verification and calibration, consisting basically of reliable measurements of stormwater flow and quality characteristics at specific points in the sewer system. These data may be compared with computed values to verify or adjust the Urban Runoff model for local conditions.

3.2.1 *Meteorological Data*

In assessing the applicability of meteorological data for Urban Runoff modelling, the critical considerations are timing, location of gauges, and network density. Analytical studies [12] have been performed employing statistical precipitation synthesis techniques to determine optimal network design. However, all too often there are basic inadequacies in the available records.

Because of the short response times and rapid flow fluctuations typical of urban stormwater runoff, short time interval rainfall data is required for detailed modelling. In most cases a 5-minute interval is sufficient, but intervals as short as one minute may be necessary for modelling very small areas. The automatic tipping bucket recording raingauge is perhaps best suited for recording such

short interval rainfall information. Each tip of the bucket (.01 inches) is recorded on a moving chart, which may be read to 5-minute intervals. Chart speeds may be increased on some instruments to provide resolution at 1-minute intervals. Weighing type recording raingauge are usually difficult to read at intervals of 15 minutes or less. In Canada, tipping bucket gauges are employed almost exclusively.

Rainfall usually varies considerably from point to point within a watershed. A good location for a single raingauge is in the centre of the study area, although for small areas, closely adjacent locations may be suitable. As the distance from the gauge to the study area increases, the error between the point rainfall measurement and the watershed average increases. This increase can be as great as 100% over a distance of only several miles [4]. For larger watersheds, it becomes even more important to have at least one raingauge located in the study area, since the effects of distance and increased area combine to reduce point measurement accuracy.

The density of the raingauge network is also an important consideration as the catchment sizes increase. For small catchments (up to 1000 acres) a single raingauge should provide a reliable information. Some of the largest catchments modelled to date have been between 3000 and 4000 acres. Significant differences in storm precipitation have often been recorded between several gauges located within watersheds of this size. The use of an individual raingauge as in these instances, may result in inaccurate simulations. Studies using dense raingauge networks on small areas indicate that use of two raingauges within the catchment decreases the error between point measured and basin average precipitation (as determined by many gauges) by as much as 50% [4]. Other factors affecting the desired density of raingauges would be the predominance of certain types of storm patterns over an area, and topographic influences such as major elevation differences. These may lead to significant errors between point measured and basin average rainfall. The desirability of using two raingauges in an area cannot be overstressed.

Temperature data may be required for the modelling of snowmelt. Variations in temperature within an urban area are much smaller than variations in precipitation. Reliable information may be obtained from the nearest meteorological station. For most snowmelt modelling, daily maximum and minimum temperatures are sufficient.

In Canada, the source of most meteorological data is the Atmospheric Environment Service of Environment Canada. Types and formats of available data are discussed in Chapter 12.

3.2.2 *Physical System Data*

Physical system data encompasses all the geometric, hydrologic and hydraulic information necessary to describe the subcatchment areas, sewer system, and the basic connectivity of these elements, for the computation of stormwater runoff flows. In addition, information on street sweeping practices, land use, population and other parameters is also necessary for computation of runoff quality. Such a data base is common to most single event models, and to a lesser extent to the more simplified continuous simulation models. Data are generally available from topographical and sewer maps, plans and profiles, aerial photos, field reconaissance information, etc., through the local Municipal Engineering office. The physical data requirements for the SWMM are fairly extensive, but are relatively straightforward and may sometimes be reduced by adopting a more simplified modelling approach as described in Chapter 10 of this report. A detailed summary of the physical data required for SWMM modelling is contained in the User's Manual.

The most frequent problems with physical data are missing, incomplete, or faulty information on basic documents such as plans and profiles. This often necessitates costly field verification of critical measurements.

3.2.3 *Verification and Calibration*

Verification and calibration data provide the means to check the operation of the Urban Runoff model, or a part of it, either during the development of its basic routines, or during the application of the model in a particular study. These data must therefore be as accurate as possible in order to play a useful role. Sufficient latitude is available in selection of certain of the model parameters in SWMM to achieve a closer agreement between the simulation and measured data for a particular watershed by calibration. Chapter 4 contains a discussion of the sensitivity of the SWMM to variations in model parameters.

3.2.3.1 *Flow Measurement*

The accuracy of flow measurements is largely dependent on the measuring device employed. Methods applicable to sewer environments may be of three types; critical depth devices, tracer or acoustic techniques, or simple stage measurement coupled with an empirical flow formula. These techniques have been reviewed elsewhere by Wenzel [5], Schontzler [6], Marsalek [7], and Lager and Smith [8]. Only a brief discussion will be given here.

Critical depth devices employ established stage discharge relationships coupled with a measurement of the stage during the flow event. Measurement of the stage may be accomplished with floats, gas bubblers, surface probes, capacitance probes, or ultrasonic devices. Accuracies of $\pm$ 5% are possible with carefully calibrated critical depth sections.

Tracer or acoustic techniques may be used to determine flow velocity directly. A knowledge of the flow area yields the flow rate. Accurate results may be obtained ($\pm$ 5%), although more sophisticated, expensive equipment is generally required.

Stage measurement coupled with an empirical flow formula (such as Manning's equation) is the least accurate of all common methods. However the simplicity and economy of this method has gained wide acceptance for small area studies and for purposes unrelated to computer modelling. These techniques are susceptible to error because of the assumption of steady, uniform flow, which is rarely the case in most sewer systems. Discrepancies of up to 20-25% must be expected with this type of measurement system.

A second basic consideration is the location of the measurement site. Aside from maintenance considerations such as ease of access, space requirements, etc., it is important to establish precisely which flow is being measured. Overflows, diversions pumping stations and storage facilities located upstream must be identified and their impact assessed regarding the measured values. The effects of backwater due to lakes, streams, or interception must also be evaluated.

The length of the historical record for a particular site may be an important factor, especially when continuous simulation is contemplated. In this case, accuracy may be sacrificed in favour of the number of recorded events, which can be most useful in estimating annual discharges.

3.2.3.2 *Quality Sampling*

The collection of accurate stormwater quality data is perhaps the most difficult aspect of a monitoring programme. An excellent assessment of currently available samplers was given by Shelley and Kirkpatrick [9]. Because no single sampler can carry out all the functions that may be required, the suitability of individual samplers should be assessed at the beginning of each programme.

Site selection is also of prime importance in establishing a sampling programme. The location should be as close as possible to the flow measuring device. Factors that can affect the representativeness of the samples are; flow velocities at the sampling point, location in the cross-section at which sampling occurred, existence of a uniform conduit section upstream, contribution of tributary inflows, and finally the type of samples collected.

Model calibration and verification usually necessitates frequent collection of discrete samples in order to determine the variation in constituents with progression of the storm flow hydrograph. Intervals of 15 minutes have been utilized in many collection programmes. The duration of the sampling process will depend on the capacity of the sampler (if it is automatic), and the requirements of the modeller. It is often important to establish the characteristics of the start, peak and end of an event. Composite samples, made up of a series of smaller samples, yield time averaged values which may not be useful for direct model comparisons. However, comparison of mass discharges of pollutant recorded over one or several storm events with computer simulations may provide a good means of assessing model performance and overflow problems. Composite samples can sometimes be quite useful, and less expensive than discrete samples, depending upon study objectives.

The number of pollutant constituents analyzed affect both the cost and usefulness of the data. For calibration/verification with the SWMM results, data describing three basic constituents are required; BOD, suspended solids, and total coliforms. These three pollutants can be modelled through in all phases of the program.

The length of sampling record is an additional factor in data assessment, since the longer the record, the more data are available for establishing stormwater conditions and model response. A longer sampling period can be especially useful when continuous simulation is planned.

3.2.3.3 *Time Synchronization*

Of some importance in any collection programme is the synchronization in time of the three measurements; rainfall, flow and sample. In the ideal situation all three would be recorded on the same chart. This is possible with some of the newer recorders available, which have been used in recent data collection programmes. In many small scale programmes, the individual data is recorded separately. Where the instruments are also physically separated, there is a possibility of questionable timing of each recording. Apparent time shifts of 10 to 15 minutes between the rainfall and flow record are usually not a significant factor, and may be accounted for in the model calibration process by adjusting the starting time. It is usually advantageous when dealing with separate recording elements to employ a control clock and apply a check to the timing of each recorder during each site visit.

3.3 CANADIAN STUDY AREAS

An investigation of a number of on-going studies, municipal programmes, and university research was carried out very early in this study to determine the availability of Canadian data suitable for modelling. Particular emphasis was placed upon Canadian data for several reasons. Firstly, conditions in Canada are well known to potential users and this study was primarily Canadian in administration and purpose. Secondly, all of the reliable American data

available at the start of this study had already been used in numerous simulations and were currently being employed in on-going American studies such as those of Brandstetter [2], Heaney [11], and others. Again, if suitable Canadian data did exist, their use would help focus attention in this country on the availability and requirement for further Canadian collection studies and the use of computer models in general. Finally, much of the American data, such as that recorded in Bloody Run, Cincinnati, and Atlanta, Georgia, was either incomplete or unreliable in one or more of the basic elements. Some of the more reliable U.S. data, such as Oakdale, Illinois, were from very small watersheds and therefore limited in scope.

Data from eight Canadian watersheds were reviewed. In some cases, the data represented purposeful efforts to collect data for modelling, whereas in other cases the collection programme was oriented towards overall assessment and monitoring studies conducted by Municipal departments. The areas reviewed are summarized in Table 3.1.

TABLE 3.1

SUMMARY OF AVAILABLE CANADIAN DATA

Area	Size (acres)	Type	Rainfall Data	Flow Data	Quality Data	Period of Record
Brucewood	48	Storm	G	G	G	'74-'75
East York	40	Storm	G	F	P	'74-'75
Malvern	58	Storm	G	G	G	'73-'75
Kingston	89	Storm	G	G	-	'73-'75
Halifax	168	Combined	G	G	G	'71-'74
Malton Airport	495	Storm	G	G	G	'74-'75
Bannatyne	542	Combined	P	G	F	'69-'71
West Toronto	2330	Combined	G	F	-	'69-'76
Montreal	2880	Combined	P	F	F	'73

G = Good
F = Fair
P = Poor

3.3.1 *Brucewood Catchment, North York* [16]

The Brucewood site provides a good source of modelling data. Many low intensity, and several medium intensity storms have been monitored for both flow and quality. Rainfall data are provided by an automatic tipping bucket raingauge located adjacent to the study area. Flow data are obtained using a weir at the outlet, providing continuous flow measurement of all events. Samples are collected automatically at 4-minute intervals from the outlet pipe during storm flows. In addition, snowmelt studies, and investigations of surface pollutant accumulation have been carried out on the test area.

Rainfall and flow information is remotely recorded on synchronized charts, making this a good source of Urban Runoff data.

3.3.2 *Barrington Catchment, East York* [19]

A limited amount of data have been collected from this area. Rainfall data were provided by an automatic gauge within the area. A flow measuring weir was used to measure the runoff hydrographs, but examination of these suggests that in many cases readings were incomplete. Most storm events were of very low intensity, and therefore not particularly useful for detailed modelling. In some cases, large unexplained timing shifts occur between recorded rainfall and runoff. Both manual and automatic sampling were employed in collecting these data. The collection times of the bacteriological samples were not shown on the lab reports and consequently these are of no use for model verification.

These data were considered to be of limited use for modelling purposes.

3.3.3 *Malvern* [24]

Very good runoff data have been collected on this area. Rainfall was monitored within the catchment, and recorded on the same chart as the runoff. Flow measurement was accomplished using a calibrated weir located at the drainage outfall. For the period of '73-'74, about 22 storms were monitored and used for runoff modelling. The runoff quality monitoring started in 1974. The project will be continued in 1976.

3.3.4 *Calvin Park, Kingston* [18]

Very good runoff data have been collected on this area. Rainfall was recorded within the catchment, and recorded on the same chart as the runoff. Flow measurement was accomplished using a weir. Only a limited number of high intensity storms were available. No quality data were obtained during this monitoring programme, but it is understood that recently the study has been expanded to include water quality measurement.

The quantity data were considered very good for modelling purposes.

3.3.5 *Halifax* [20,21,22,23]

Good data have been collected in the Halifax area in two programmes; the first in 1970-71 monitoring combined storm flows, and the second in 1974 recorded winter runoff. Rainfall data are excellent, supplied by an automatic recording raingauge located within the area, supported by several standard raingauges. Flow was recorded using a weir at the inlet to an overflow retention tank. Water quality was sampled both automatically and manually at the outlet. Most of the recorded storms are of very low intensity, and only a few high intensity storms are available.

These data are considered suitable for modelling purposes. However, other independent studies were already using the best data from this area, and repetition of these efforts was considered unnecessary.

3.3.6 *Bannatyne, Winnipeg* [17]

A large amount of data has been collected from several urban catchments in Winnipeg. The most complete data from the Bannatyne combined sewer area, were collected from 1969-1971. Originally, recording raingauges located within the study area were available, but these records were misplaced by the collecting agency. Data from three meteorological stations located outside the study area are available. Flows were measured using a weir, and flows for a large number of complete storm events were recorded. However, a

limited number of events for which both rainfall and flow data are available have been identified. An automatic sampler was used to collect water quality samples, and data are available describing quality during part of each of the events. These data were recorded separately from the flow charts with some unexplained time shifts occurring.

Some of these data were considered to be suitable for modelling. The weakest element of the collected data was the rainfall, but reasonable agreement between the nearby raingauges was evident for several of the recorded events. Quality data were incomplete in some events due to the short operating time of the sampler used, but in view of the state of the art of quality modelling, these data were also considered to be of some use.

3.3.7 *Toronto International Airport* [13]

Stormwater monitoring was carried out for several areas on the Malton Airport site, during 1974 and 1975, as part of an assessment of environmental problems. One of the larger catchments was an area of 495 acres consisting of level grassy area (52%) and of flat impervious surfaces such as runways and maintenance buildings (48%).

Flow levels were measured using a Manning "Dipper" level recorder located in a 72-inch outfall sewer. Flow was determined using Manning's formula. An automatic recording raingauge was located nearby at the airport meteorological station. Data from several storms are available for this catchment and were considered to be suitable for modelling purposes. (Subject to the limitations of the flow recording technique used) Quality sampling was conducted using automatic samplers as the outfall to this area.

3.3.8 *West Toronto* [14]

The West Toronto watershed has been monitored for overflow quantity for the last 6 years. The data collected for this large area were of interest both for detailed single-event modelling and for continuous modelling. Very good rainfall data are available for this area, from a recording raingauge operated by the Atmospheric Environment Service, located approximately in

the centre of the area. The area is large enough to warrant additional rainfall information, and fortunately correlation with another A.E.S. raingauge located outside the boundary of the area was possible. The area drains to a large outfall sewer in which water levels have been recorded during all storm events. An examination revealed that no backwater conditions affected the level measurements. However, a complication was a series of diversion weirs at the outlet designed to intercept dry weather flow in sewers leading to the outfall. However, it was possible to re-construct an approximate outflow hydrograph, by computing uniform flow from the level measurements, and adding an estimated component representing flows diverted both at the outlet, and at a number of minor points along the upstream boundary. Many major overflows have been recorded at the outfall pipe (108" dia.) where the capacity of the intercepting sewers forms only a small proportion of the outflow. While flow measurements were recorded in the major intercepting sewer during the period in which overflows were recorded, recent measurements are available to more precisely establish the intercepted flow. No quality measurements were available from this area.

Overall, the quantity data from West Toronto were judged suitable for modelling purposes. The inherent inaccuracies in the flow determination are noted, but there are important aspects to the Toronto data. It is a large area (2330 acres), served by combined sewers and prone to surcharge. Few such areas, which are typical of many large city areas, have been modelled using the SWMM. The length and continuity of the record made it valuable for continuous simulation, in which case point accuracy of individual peak flows was of less concern than the frequency, duration and total volume of overflow. The area had been previously modelled in detail with the Dorsch Hydrograph-Volume Method (HVM), and consequently a considerable amount of physical data was available in reduced form. This also presented an opportunity to perform an approximate comparison between these two models.

3.3.9 *Papineau-Curotte Catchment, Montreal* [15]

Limited measurements are also available for a large area in Montreal. Rainfall data are available, but none of the recording raingauges are located

within the area. Flow is measured at the outlet using a weir, and data are available for 10 events of relatively low intensity rainfall. Quality data are available from automatic sampling of only 4 of these events. It is not clear exactly how the flow measurements were obtained since this area also drains to an interceptor/overflow system. The quality sampling was conducted some distance downstream of the study area, and so the contributing area is uncertain.

The data from this area were considered to be unsuitable for the present modelling purposes.

3.3.10 *Current Collection Programmes* [10]

Under the sponsorship of Environment Canada, two urban areas have been instrumented for collection of complete urban runoff data. This programme is specifically designed to provide data for runoff modelling.

The first of these is a 176 acre area, located in Hamilton, served by a combined sewer system. This area is mainly single-family residential, and is about 45% impervious. Rainfall is being measured with two tipping bucket raingauges, located within the watershed. A modified v-notch weir and a bubble type level sensor are utilized for flow measurement. Automatic quality sampling is being carried out. The data from these elements is being logged simultaneously on a single recorder device. In addition, data on road and catchbasin cleaning, snow removal and de-icing, and temperature are being collected.

A second test area, located in Toronto, has also been instrumented for urban runoff data. This area covers 338 acres of predominantly single-family residential land use. The area is 49% impervious. Precipitation measurement is by a single tipping bucket raingauge and a special high resolution recorder. Flow measurement apparatus consist of a bubbler type level recorder and a partial trapezoidal weir. Automatic sampling is done during storm events, initiated by a pre-determined water level. Continuous temperature measurement is carried out at the outfall site, and information

on municipal cleaning practice is also being collected.

Both of these measurement programmes have been in operation since the late summer of 1975. These are potentially excellent sources of runoff data, specifically oriented towards runoff modelling.

3.3.11 *Summary of Areas Reviewed*

Several sampling programmes were identified as being potentially suitable for modelling purposes within the context of the present study. Data were being collected from the Brucewood area which would be available for inclusion in this study. The only large areas with acceptable data were in Winnipeg and West Toronto. It was considered that the less accurate Toronto data would be very useful in portions of the study that did not include detailed model flow verification, such as simplified simulation and long term simulation. The Winnipeg data represented a medium sized watershed with good records of quantity and quality, which would be useful in various parts of this study. Finally, limited data from the Malton Airport and Barrington watersheds were considered to be suitable for some study purposes, and for general quantity simulation.

A review of data available from U.S. watersheds is contained in Volume 2. Data from the Lancaster area and the Atlanta area were obtained for use in this study. Although these data were considered the best available in the U.S. at the start of this study, there were some inadequacies. Lancaster flow measurements were considered somewhat unreliable, and the absence of good rainfall data for the Atlanta area had resulted in poor simulations in previous work.

3.4 CONCLUSIONS

(a) Based upon a review of available data, it was found that complete sets of measured storm runoff data were scarce both in Canada and the U.S.A. Most of the acceptable data sets which have been published are for very small catchments, and have been modelled many times in previous studies.

(b) Several sets of data from Canadian Urban watersheds were identified as being suitable, at least in part, for modelling within the terms of the study.

(c) There is a continuing need for more runoff data for Canadian watersheds from programmes specifically designed for modelling needs. Existing monitoring programmes should be continued.

(d) The measurement of rainfall is often overlooked in setting up monitoring sites. In general, at least one automatic recording raingauge should be located in or immediately adjacent to the monitored watershed. For large catchments, greater than 1000 acres, two raingauges should be employed.

REFERENCE - CHAPTER 3

1. U.S. Department of Health, Education and Welfare, "Pollutional Effects of Stormwater and Overflows from Combined Sewer Systems", Public Health Service Publication No. 1246, November 1964.

2. Brandstetter, A., "Assessment of Mathematical Models for Storm and Combined Sewer Management", Preliminary Report, Battelle Pacific Northwest Laboratories, Richland Washington, August 1975.

3. Huber, W., et al, "Establishment of an Urban Rainfall-Runoff Data Base", Interim Report Phase 1, Department of Environmental Engineering Sciences, University of Florida, Gainesville Florida, March 1975.

4. U.S. Department of Agriculture, Soil Conservation Service, "National Engineering Handbook Section 4 - Hydrology", Washington, D.C., August 1972.

5. Wenzel, H.G., Jr., "A Critical Review of Methods of Measuring Discharge Within a Sewer Pipe", Technical Memorandum No. 4, ASCE Urban Water Resources Research Program, American Society of Civil Engineers, New York, 1968.

6. Schontzler, J.G., "New Electronic Flow Measurement for Wastewater", in "Proceedings of the International Seminar and Exposition on Water Resources Instrumentation", Krizek, R.J., and Masanyi, E.F., eds., Ann Arbor Science Publishers, Inc., Ann Arbor, Michigan, 1975.

7. Marsalek, J., "Instrumentation for Field Studies of Urban Runoff", A Hydraulics Division Research Report, Canada Centre for Inland Waters, Burlington, Ontario 1975.

8. Lager, J.A., Smith, W.G., "Urban Stormwater Management and Technology: An Assessment", U.S. Environmental Protection Agency Document EPA-670/2-74-040, Cincinnati, Ohio, 1974.

9. Shelley, P.E., Kirkpatrick, G.A., "An Assessment of Automatic Sewer Flow Samplers", Office of Research and Monitoring, U.S. Environmental Protection Agency, Document No. EPA-R2-73-261 Washington, 1973.

10. Department of the Environment, Canada Centre for Inland Waters, "Storm Water Management Model Verification Study", Current.

11. Heaney, J.P., Huber, W.C., et al, "Urban StormWater Management and Decision Making", Environmental Protection Technology Research Series, EPA-670/2-75-022, U.S. Environmental Protection Agency, Cincinnati, Ohio, May 1975.

12. Eagleson, P.S., "Optimum Density of Rainfall Networks", Water Resources Research, Vol. 3, No. 4, 1967.

13. James F. MacLaren Limited, "A Study of Environmental Problems at Toronto International Airport", Prepared for the Environment Protection Service, Ontario Region, July 1975.

14. Department of Public Works, City of Toronto, Personal Communication with Mr. C. Kitchen, August 1974.

15. Service D'Assainissement des Eaux, Division des Usines, Communaute Urbaine de Montreal, Personal Communication with Mr. Urgel Bechard, ing., chef de groupe, December 19, 1974.

16. Unpublished Report to Canada Department of the Environment and the Ontario Ministry of the Environment, Prepared by James F. MacLaren Limited, 1975.

17. Unpublished Report, City of Winnipeg, Waterworks and Waste Disposal Division, 1971.

18. Kidd, C.H., "The Development of an Urban Runoff Model", Unpublished N.S. Thesis, Department of Civil Engineering, Queen's University, Kingston, 1972.

19. Engineering Department, Borough of East York, Personal Communication with Mr. G. Mills, 1974.

20. Waller, D.H., "Pollution Attributable to Surface Runoff and Overflows from Combined Sewerage Systems", Central Mortgage and Housing Corp., April, 1971.

21. Waller, D.H., Personal Communication.

22. Waller, D.H., et al, "A Comparative Evaluation of Two Urban Runoff Models", Unpublished Report to Water Pollution Control Directorate, Environment Canada, April 1974.

23. Waller, D.H., and Coulter, W.A., "Winter Runoff", Report Prepared for the Water Pollution Control Directorate, Environment Canada, May 1974.

24. Marsalek, J., Canada Centre for Inland Waters, Burlington, Personal Communication.

SWMM FLOW SIMULATION FOR SELECTED CANADIAN WATERSHEDS

CHAPTER 4

CHAPTER 4

SWMM FLOW SIMULATION FOR SELECTED CANADIAN WATERSHEDS

4.1 GENERAL

Accurate flow simulation is a prerequisite for most studies of storm drainage and pollution abatement. The first comparisons of flow simulations given by different urban runoff models were carried out by those involved in the construction of the models and their scope was mainly confined to calibration and verification, based on only a few events. Sixteen different flow simulation models are discussed in a recent study conducted by Battelle Pacific Northwest Laboratories [1].

A study comparing different flow simulation models, was sponsored by Canada Centre for Inland Waters [2]. This study investigated a much larger number of storm events than other previous model comparisons, and the models were evaluated from the standpoint of a potential user rather than on a strictly theoretical basis. Small urban catchments (less than 100 acres) were employed for these comparisons. The urban runoff models studied in detail were limited to non-proprietary, single-event, runoff simulation models. The following runoff models were selected on the grounds of availability, technical sophistication, and recent successful application.

(a) Road Research Laboratory Model (RRLM)
(b) Storm Water Management Model (SWMM)
(c) University of Cincinnati Urban Runoff Model (UCURM)

In addition a limited number of simulations was carried out using the Dorsch HVM (Hydrograph Volume Method) model and the Unit Pulse model developed at Queen's University.

The main conclusion of the study was that in general flows simulated by all urban runoff models agreed fairly well with measured runoff hydrographs for small urban catchments. A statistical comparison of the simulated hydrographs with the measured flows resulted in good agreement for SWMM, fair to good for RRLM and fair for UCURM. These results are summarized

in Table 4.1. Of the non-proprietary runoff models investigated, SWMM gave the best results. Since it is in general a more versatile and better documented model, the SWMM was recommended for subsequent work on larger areas.

In large urban areas routing in the trunk sewer system plays a significant role in determining the outflow hydrograph. In the framework of the present study the SWMM quantity model was assessed by the simulation of storm flows on large watersheds by both RUNOFF and TRANSPORT Blocks, and results were compared with measurements and those of other more sophisticated routing models.

TABLE 4.1

A COMPARISON OF RUNOFF MODELS [2]

	(COMPUTED PEAK FLOW)/ (MEASURED PEAK FLOW)			(COMPUTED TIME TO PEAK FLOW)/(MEASURED TIME TO PEAK			(COMPUTED RUNOFF VOLUME)/(MEASURED RUNOFF VOLUME)		
MODEL	No. of Observations	Average	Standard Deviation	No. of Observations	Average	Standard Deviation	No. of Observations	Average	Standard Deviation
RRL MODEL (Non-Calibrated)	57	1.09	0.25	57	1.03	0.23	20	1.09	0.27
SWM MODEL (Non-Calibrated)	58	1.13	0.23	58	0.95	0.22	20	0.96	0.22
UCUR MODEL (Non-Calibrated)	27	1.15	0.23	27	0.86	0.14	10	0.83	0.14
SWM MODEL (Calibrated)	31	1.01	0.21	31	0.96	0.11	10	1.03	0.17

4.2 BASIC CONCEPTS

The urban watershed drainage system (Figure 4-1) may be considered as two distinct systems for computer modelling purposes.

(a) The basic subcatchment system in which the excess rainfall is transformed to overland flow. The overland flow hydrographs at the drainage manholes form the inlet hydrographs for the transport system.

(b) The transport system in which the flows are routed through the trunk sewer network.

4.2.1 *The Basic Subcatchment System*

Each subcatchment is conceptualized as a flow plane over which overland flow occurs. Representative values of imperviousness, average ground slope, infiltration, retention storage, and land use are assigned to each subcatchment. The rainfall depth over each subcatchment is the basic input to the overland flow or RUNOFF model. The model sequentially accounts for infiltration and retention depth to determine a net excess rainfall depth on the overland flow plane. The excess rain depth is used to compute an overland flow rate for each time step. The SWMM uses a very simple algorithm for this runoff computation as shown in Figure 4-2. Subcatchments can be used to represent large portions of the entire watershed, or individual drainage areas, depending upon the desired degree of detail. The small local sewer pipes and gutters are usually ignored in large subcatchments. The SWMM also provides the facility to account for simplified routing in local pipes in the RUNOFF Block. Instead of considering the subcatchment overland flow hydrograph to flow directly to the inlet manhole, it can be routed initially through a "GUTTER" subroutine which is used to account for the many ditches, lateral pipes and street gutters, which are not usually included in the TRANSPORT Block computations.

4.2.2 *Routing Models*

A routing model combines the runoff flows from several subcatchments and routes these flows through the main sewer network. Several urban runoff models of varying levels of sophistication have been developed for the simulation of this process. Such TRANSPORT models range from those using simple time-offset methods to those employing complex numerical schemes for the solution of the dynamic wave equations. While the latter approach is applied by some one event simulation models (WRE model, Dorsch HVM), long term simulation models generally use simpler techniques and others do not consider sewer routing at all (e.g. STORM). A comprehensive TRANSPORT model should be able to account for the following:

*(a) The unsteady and non-uniform nature of flow through sewers.

*(b) The storage capacity of the sewer system.

(c) The dynamic effects of the backwater condition and the energy losses along the sewers and at manholes.

*(d) Control elements including manholes, weirs, orifices, pump stations, storage units and gates.

*(e) The distribution of flow through branched and looped systems.

(f) Flow reversal and adverse sewer grades.

*(g) The variety of conduit shapes commonly encountered in engineering practice.

In order to assess the TRANSPORT Block of the SWMM, the response of the model was compared to that of two other models under varying physical and hydrological conditions. The routing models considered in this phase of the study are described below. The results of comparing these different models are described in Section 4.4 of this chapter.

---

* Accounted for in SWMM

4.2.2.1 *SWMM TRANSPORT Block*

The TRANSPORT Block of the SWMM [3] routes the stormwater flows through a converging branch sewer network. Dry weather flows and infiltration into the sewer system are computed. Two types of flow diversion structures, two storage basins and one lift pumping station can be modelled. The model is limited to the simulation of single storm events and to a maximum of 160 hydraulic elements. Larger systems may be handled by modelling the major areas separately. The flow routing is based on the quasi-steady dynamic wave approximation, this being a form of the St. Venant equations (the momentum equation for a dynamic wave), with the omission of the local acceleration term (equation 4.1). While an implicit scheme is used to solve for continuity, (equation 4.2), an explicit scheme has been adopted for the solution of the momentum equation.

$$\frac{\partial h}{\partial x} + \frac{v}{g}\frac{\partial v}{\partial x} = S_o - S_f \qquad (4.1)$$

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (4.2)$$

where h = depth
x = longitudinal coordinate along channel bottom
t = time
v = velocity
g = gravitational acceleration
Q = flow rate
S_o = channel bottom slope
S_f = friction slope
A = cross-sectional area of flow

A Newton-Raphson technique is applied for the solution of the non-linear continuity equation. The friction slope is evaluated from Manning's formula and lateral flow is not simulated. The solution procedure basically

follows a kinematic wave approach, in which disturbances are allowed to propagate only in the downstream direction. As a consequence, backwater effects are not modelled beyond the domain of a single conduit, and downstream conditions will not affect upstream elements in the normal TRANSPORT computations. However, backwater can be approximated at a maximum of two locations by specifying a storage element and providing appropriate geometric input data. These effects are, again, not transmitted to the upstream elements. The program does not simulate pressure flow conditions. Surcharging is modelled simply, with flows in excess of the full flow conduit capacity stored at the upstream manhole until capacity exists to accept the stored volume. Only tree shaped systems are permitted. Accordingly, interconnections, looped networks, and flow reversal cannot be simulated. A maximum of three inlet and one outlet conduits can meet at any manhole. The geometries of 12 commonly used closed conduit shapes are programed internally, and three additional arbitrary shapes may be specified by the user.

4.2.2.2 *Dorsch Hydrograph Volume Method (HVM)*

The HVM Model [1,4,5,6] simulates the system as an interdependent network and the effect of conditions in each element (backwater effects, surcharging and pressure flow conditions) are modelled. The model accounts for combined and storm sewer systems, consisting of loops and converging and diverging branches and open channel networks. The flow routing is based on an implicit finite difference solution of the energy and continuity equations (4.3, 4.4), which are applied at each node and along each sewer segment.

$$\frac{\partial h}{\partial x} + \frac{\partial}{\partial x}\left(\frac{V^2}{2g}\right) + \frac{1}{g}\frac{\partial V}{\partial t} + \frac{q}{gA}\left(V - V_q \cos\alpha\right) = S_0 - S_f \qquad (4.3)$$

$$\frac{\partial Q}{\partial x}dx + \frac{\partial A}{\partial t}dx = q\,dx \qquad (4.4)$$

where
- h = depth
- x = longitudinal coordinate along channel bottom
- t = time
- v = velocity
- g = gravitational acceleration

q = lateral inflow
= junction interior angle
S_o = channel bottom slope
S_f = friction slope
A = cross-sectional area of the flow
Q = flow rate

An iterative solution technique is employed, applying a repetitive downstream and upstream computational scheme. A maximum of three inflow and two outflow conduits can come together at a node. A maximum of 1100 sewer segments can be accommodated in one calculation area. The model simulates weirs, orifices, gates, diversion structures, retention and drop structures. The surface runoff inflow enters the conduit elements laterally from the surface runoff model. The model, like SWMM also simulates dry weather flow. This proprietary model has been applied extensively.

4.2.2.3 *WRE Model TRANSPORT Block*

The WRE model [7,8] is a modified version of the SWMM. The TRANSPORT Block has been improved, and the new model has additional capabilities. The dynamic wave equations are solved by a special explicit finite difference formulation. The continuity equation (4.5) is applied at the nodes (sewer junctions and connections) and the momentum equation (4.6) is applied along the links (sewer and channel reaches).

$$\left(\frac{\partial H}{\partial t}\right)_t = \frac{\sum Q}{As_t} \tag{4.5}$$

$$\frac{\partial Q}{\partial t} = -g\ AS_f + 2V\ \frac{\partial A}{\partial t} + V^2\ \frac{\partial A}{\partial x} - g\ A\ \frac{\partial H}{\partial x} \tag{4.6}$$

where $\sum Q$ = net inflow to the junction
Q = flow rate in the conduit
V = velocity
A = cross-sectional area of the flow
A_s = surface area of the node (manhole)
H = hydraulic head
S_f = friction slope
t = time

The sewer system is simulated as an inter-dependent network and the effect of each element on the system is modelled. This is achieved by applying a Newton-Raphson iterative technique in solving the whole system as one unit at each time step. Surcharging conditions can be modelled at the surcharged junctions. An approximate first order correction based on the Hardy Cross method is applied independently at the surcharged junctions. When the hydraulic grade line intersects the ground surface, the excess flow volume is lost from the system. A maximum of eight conduits can join at a node and up to 320 pipes are modelled. The model allows for flow reversal and consequently pipes are not specified as inlets or outlets to a manhole. Looping effects and converging and diverging branches can also be simulated. The model allows for the geometries of five closed conduit shapes and a trapezoidal open channel. Four hydraulic control devices are allowed, namely, weirs, gates, orifices and pumping stations. As in the SWMM, all flow enters at junctions and no lateral flow is considered.

4.3 SWMM SENSITIVITY ANALYSIS FOR SELECTED INPUT PARAMETERS

An appreciation of the sensitivity of the SWMM computations to changes in input data is important in setting up and conducting detailed simulations. This is especially true when some of the data are unavailable or unreliable, and a decision must be made to proceed with a simulation or obtain more basic data. Because of the large data requirements, additional collection may be a costly and time consuming process (such as obtaining additional topographic mapping). A limited sensitivity analysis was conducted on the main RUNOFF and TRANSPORT Block parameters as a part of this study. The purpose was to increase familiarity with the model, and to establish the most sensitive parameters before conducting detailed simulations of the West Toronto and Winnipeg areas.

4.3.1 *Runoff Block Sensitivity*

A review of the basic computational theory upon which the RUNOFF Block is based indicated the following parameters would directly affect the surface runoff from a given area:

1) Imperviousness
2) Width of overland flow
3) Infiltration
4) Retention depth
5) Manning's 'n'
6) Ground slope

Simulations were performed in which these parameters were varied over practical ranges and the results compared with those obtained using the default values built into the SWMM. A hypothetical watershed consisting of 25 small 11 acre subcatchments was employed in these sensitivity simulations. A one hour triangular rainfall pattern with a peak intensity of 4 inches per hour and a total volume of 2 inches was applied. By varying parameters on each of the identical subcatchments, numerous sensitivity analyses could be performed in one program run. These sensitivity simulations were all run for a duration of 200 minutes, which is adequate to simulate the runoff hydrograph from a 60 minute rainfall event. At the end of a simulation, there is usually some amount of surface water storage left on the catchment which would eventually run off. Changing any parameter which delays runoff from the subcatchment, such as ground slope 'n', or "width" results in a greater amount of surface storage remaining at the end of simulation, and therefore affects the volume of runoff during the simulation. This left-over storage would eventually run off, however, if the simulation were conducted for an indefinite period. The following discussions consider the actual period of simulation, and for this reason parameters which do not affect water losses, such as 'n', will have an effect on runoff volumes.

4.3.1.1 *Sensitivity to Infiltration Rates*

Figure 4-3 shows the result of the variation of infiltration parameters on the peak flow and volume of surface runoff. Different rates of maximum and minimum infiltration were investigated, all using the default value for the decay rate of infiltration with time. The rates were varied in multiples of the default values (3.0"/Hr maximum, .52"/Hr minimum). The per cent imperviousness for all three cases was 50%.

The results indicate that the peak flow is not very sensitive to the infiltration rates since only an 8% variation in peak flow was observed. A somewhat greater effect was observed on total runoff volume, which varied some 25% over the range of rates applied. For areas with a greater amount of pervious surface, the peak flow and volume would be more sensitive to infiltration, and the converse would be true for highly impervious areas. For lower intensity storms, during which all of the rainfall infiltrates into the pervious area, the runoff would be less sensitive to infiltration rates, unless these were reduced enough to allow an excess rainfall to develop on the pervious areas.

4.3.1.2 *Sensitivity to Retention Depth*

Surface retention depths are specified for the pervious and impervious areas modelled. The default values are .062" and .184" respectively. Four sets of retention depths were simulated, ranging from 0 to 4 times the default values for imperviousness and pervious areas.

The results are shown on Figure 4-3. The sensitivity observed was similar to that for the infiltration. Peak flow was relatively insensitive with a variation of 9%, and volume of runoff only moderately sensitive with a total variation of 20%. The similarity to the infiltration results was anticipated since both parameters act in the same way to modify the rainfall excess depth over the subcatchment.

4.3.1.3 *Sensitivity to Ground Slope*

Figure 4-3 shows the results of varying the subcatchment ground slope from .001 to .10. At higher slopes (greater than 1%), there appears to be a levelling off in the curves indicating reduced sensitivity. For flatter slopes, there is considerable sensitivity of peak flow to the ground slope. Runoff volume also shows greatest sensitivity at lower slopes, but this is attributed to a greater buildup of water on the subcatchment surface which would drain off slowly if the simulation were continued for a longer time. As the ground slope is increased runnoff occurs faster with the result that a greater runoff volume has occurred by the end of the simulation. If the simulation were run for an indefinite period, the volume of runoff would be the same for all slopes.

4.3.1.4 *Sensitivity to Manning's 'n'*

Manning's 'n' values for pervious and impervious surfaces were varied in multiples of the default values as shown on Figure 4-3. Both volume and peak flow increase as 'n' is reduced, and the sensitivity is greatest for low values. This is explained by the fact that lower 'n' values permit faster runoff from the subcatchment, and consequently less surface storage at the end of the simulation. For much longer simulations, runoff volume would be independent of 'n'.

4.3.1.5 *Sensitivity to Imperviousness*

The per cent imperviousness of a subcatchment has an almost linear effect on computed peak outflow and volume. The outflow increases as the proportion of impervious area increases. For very low values of imperviousness, the outflow would be primarily dependent on pervious area runoff alone. Figure 4-3 shows the variation of outflow rate and volume with imperviousness. Graham et al [9] also found this to be the most influential surface runoff parameter.

4.3.1.6 *Sensitivity to Width of Overland Flow*

The width of overland flow was varied from 5% of its correct value (1414 ft. for the hypothetical catchment), to 400% of this value. This parameter appears to be less sensitive at higher values than at low ones. Both peak flow and volume show similar effects. The width is actually a measure of the ease with which surface runoff occurs. Because the continuity equation is solved on the surface of the subcatchment in terms of rainfall, surface storage and outflow, the resulting sensitivity is non-linear. Thus if the width is reduced by one half, the outflow rate for a given depth is also reduced, but more excess water builds up on the catchment surface which tends to increase the outflow. This effect (further discussed in Chapter 9) is employed to provide compensating storage when conduits are neglected in simplified simulations. The outflow volume is reduced as the width is decreased, because it requires a greater depth of excess water on the flow plane to discharge the same quantity of runoff. Therefore, at the end of simulation, a greater excess water depth is left on the surface.

4.3.2 *Transport Block Sensitivity*

A series of conduits and a single 11 acre subcatchment were used for the sensitivity analysis for the TRANSPORT Block. Several variations of conduit lengths, number of manholes and different total conduit lengths were investigated. This test configuration is illustrated on Figure 4-6.

4.3.2.1 *Sensitivity to Conduit Length*

The same inlet hydrograph was routed through 5 conduits whose lengths varied from 500' to 10,000'. This was a test of the consistency of the TRANSPORT model, and the results are shown on Figure 4-4. There was a logical increase in routing effect, attenuation of flow peak and a delay in the time to peak, as the conduit length increased.

4.3.2.2 *Sensitivity to the Number of Conduits in a given Length*

Simulations were made in which an identical inlet hydrograph was routed through a fixed length, made up of varying numbers of conduits and manholes. For example, the hydrograph was routed through total lengths of 1000', 2000' and 4000' of pipe, alternatively composed of 4, 2 and 1 conduit sections separated by manholes. The results are shown on Figure 4-5.

Figure 4-5a shows little difference if the hydrograph is routed through four sections of 250' or a single section of 1000'. Both the peak flow and volume are almost identical. On Figure 4-5b, a slight increase in peak flow is observed when only a single 2000' pipe is used, compared to four sections of 500'. On Figure 4-5c, an increase in peak flow was observed as more conduits were used in the routing. The peak flow is approximately 16% less with a single conduit of 4000' than with 4-1000' pipes. These results tend to support the maximum conduit length, of 4000'-5000', recommended in the SWMM User's Manual. Use of lengths greater than this will result in less accurate routing computations due to numerical instability of the finite difference technique used in the solution of equations 4.1 and 4.2

4.3.2.3 *Effect of Surcharge*

In order to investigate the effect of surcharge, a simulation was conducted with the central pipe in a five conduit sequence deliberately reduced in size to produce surcharging. The results are shown on Figure 4-6. Conduits 307 and 308 upstream of the surcharged section produced identical hydrographs to those produced when no surcharging was modelled. The hydrograph in conduit 309 which surcharged, has been truncated at the pipe capacity, and the excess volume stored at the upstream manhole for later release. The hydrographs in conduits 310 and 311 are similar to that in the surcharged pipe, but the effect is delayed according to the travel time to each one. These results are consistent with the routing procedure employed in the SWMM.

4.3.2.4 *Sensitivity to Pipe Slope*

Simulations were made in which the overland flow hydrograph was routed through 1000 ft. of conduit with slopes varied sequentially from .1% to 10%. Very little sensitivity was observed in the peak flow and volume of runoff after the routing. The hundredfold change in slope produced only a 3% difference in peak flow, and a .2% change in volume, as shown in the following table. This was a limited case, however, in which conduit routing effects were small in relation to overland flow routing effects because the pipe used was short. A greater sensitivity would be expected where longer pipes were used.

TABLE 4.2

SENSITIVITY TO CONDUIT SLOPE

	Peak Flow in cfs	Runoff Volume in $ft.^3$
Inlet Hydrograph	10.269	23256.9
after .1% conduit	9.932	23303.7
after .5% conduit	10.087	23258.1
after 1% conduit	10.166	23255.1
after 10% conduit	10.238	23256.3

4.3.2.5 *Sensitivity to Manning's 'n'*

Variations in Manning's 'n' produced more significant changes in the resulting hydrograph than did either pipe slope or conduit length. Roughness values from .013 (the default value) to .052 were used in this simulation, and the results are shown on Figure 4-7. As the roughness coefficient is increased, the routed hydrograph is retarded and attenuated. The total volumes routed through the conduits were virtually identical for all values of 'n'. The surface hydrograph is also shown in Figure 4-7 for comparison.

4.3.3 *Conclusions and Sensitivity Analysis*

The percentage of imperviousness has the most significant effect on surface runoff of all of the parameters tested. Both the peak flow and volume of runoff are almost directly proportional to the amount of impervious area. The remaining parameters are ranked in order to their decreasing influence.

- Width of overland flow
- Infiltration rates
- Retention depths
- Roughness coefficients
- Ground slope

The SWMM default values for these parameters which are shown on Figure 4-3, lie on the relatively flat portions of the sensitivity curves, and appear to be reasonable estimates when no actual site measurements exist.

The maximum length of a single conduit modelled in TRANSPORT should be limited to 4000 to 5000 feet. Use of longer conduits will result in inaccuracies in the routing computations. Subject to this limitation, the number of conduits used to model a particular uniform section of an actual pipe appears to be fairly flexible. This analysis indicates that, over a single reach, the outflow hydrograph is not very sensitive to changes in conduit slope, as long as the conduit does not become surcharged.

The roughness coefficient used in the TRANSPORT routing can have a significant effect on the outflow hydrograph, and within the limits of hydraulic experience, should be considered to be a calibration parameter.

4.4 SWMM FLOW SIMULATION

Simulations were performed on several test areas for which adequate stormflow measurements were available. All simulations were conducted in a detailed fashion, with each catchment being sub-divided into a large number of subcatchments, and the major sewer elements were modelled in the TRANSPORT Block. The default values were used for retention, infiltration and surface roughness.

4.4.1 *Bannatyne, Winnipeg*

This 542-acre combined sewer and area was subdivided into 41 subcatchments (Figure 4-8) for detailed simulation. As previously noted, rainfall data were not available for the area itself, but were obtained from three surrounding gauges which were a considerable distance from the site. A weighted average procedure based upon distance to the area was applied to the records of these gauges for each event considered. Six medium to large storm events, for which reasonable agreement existed between gauge records, were selected for simulation. It was expected that the absence of rainfall data measured in the watershed would be a source of discrepancy between computed and measured hydrographs. However, the results of these simulations discussed in the following paragraphs show a reasonable agreement.

Storm of June 19, 1971

In this simulation (Figure 4-9), the computed hydrograph agreed quite well with the measured one. The three peaks were all simulated, although the initial computed peak was about 16% lower than measured. The shape of the computed hydrograph was very similar to the measured one. The computed volume of stormwater was about 7% less than measured, representing only .01" over the entire watershed. The time to peak of the simulated hydrograph agreed well with the measured hydrograph for the first peak, and was slightly less than measured for the second and third peaks.

Storm of July 3, 1971

In this simulation (Figure 4-10), the computed peak flow and time to peak were similar to those of the measured hydrograph. The simulated peak was approximately 16% less than the measured. The total volume of runoff was only about 50% of the measured volume, representing .07" of rainfall on the watershed. This was attributed to error in the rainfall data. For such a short rainfall event, differences between the weighted average

of three gauges and the actual basin rainfall could be large enough to account for this discrepancy. The shape of the rising and falling limbs of the computed hydrograph were quite similar to those of the measured hydrograph.

Storm of July 15, 1971

Both peak flow and volume of runoff were well simulated in this double event (Figure 4-11). The rounded, flat shape of the observed hydrograph was closely simulated by the model, even during the rain-free period between the major rainfall pulses.

Storm of July 17, 1971

The computed peak flow (Figure 4-12) was within 19% of the measured peak, but the marked double peak in the observed hydrograph was not well defined in the simulated hydrograph. This may be attributed due to the dampening effect the weighted average procedure produced on a storm of this magnitude with such sporadic bursts of high intensity rainfall. The computed volume was 35% less than measured, representing a difference of .04" of rainfall on the watershed. The shape and timing of the rising and falling limbs were similar to those of the measured hydrograph.

Storm of July 28, 1971

In this simulation (Figure 4-13), the computed peak flow almost exactly corresponded to the observed value, and occurred only 10 minutes later than the observed. The computed volume was 23% less than observed, representing .02" of rainfall on the watershed. The sharply peaked computed hydrograph did not closely resemble the flatter, broader observed hydrograph. It was felt that errors in the rainfall hyetograph were responsible for this discrepancy.

Storm of September 5, 1971

In this simulation (Figure 4-14), the peak flow agreed very well with the measured value, but was slightly advanced in time. Surcharging was computed by the model in some of the upstream conduits. The SWMM model simply stores excess pipe flows in the upstream manhole of each surcharged conduit, and releases them later or when pipe capacity becomes available. This explains the rapid decay of simulated flow after the peak, and the levelling off at between 20 and 40 cfs as the surcharged flow is released to the outlet. This simulation illustrates that even with a limited TRANSPORT routine, the SWMM can give a reasonable good result under moderate surcharging.

4.4.2 *West Toronto*

For the detailed simulations, this large 2330 acre catchment was subdivided into 45 subcatchments (Figure 4-15). Rainfall data were available from a raingauge within the catchment. These were averaged with the data from another gauge outside the area, since it was felt that a single gauge could not adequately represent so large an area. As discussed in Chapter 3, the accuracy of the flow measurements is only fair, and it was necessary to adjust the overflow measurements to allow for intercepted and diverted flow volumes. Nevertheless, it was felt that this estimated flow could be used for comparative purposes. All of the comparisons discussed below represent total simulated storm flow versus the corrected total storm outflow. Five rainfall events were selected for the detailed simulations, representing a range of storm intensities for which adequate measurements existed.

Storm of June 22, 1973

The simulated flows agree very well with the observed hydrograph (Figure 4-16). The computed peak was slightly less than observed, as was the computed volume of runoff. Default values

were used, except for the percentage of impervious area with zero detention, which was increased to 45% to advance the rising limb. The computed volume was about 16% less than the observed.

Storm of September 23, 1973

The simulated peak flow and volume of runoff agree reasonably well with the observed values, although the computed hydrograph lags by about 15-20 minutes (Figure 4-17). This may be attributed to rainfall recording inaccuracies.

Storm of October 2, 1973

In this simulation, a good agreement between measured and observed was obtained for both peak flow and volume of runoff (Figure 4-18). The simulated values were only 5% less than those observed. Default values were used in this simulation. The shapes of both hydrographs are also quite similar.

Storm of August 1, 1973

Extensive surcharging in upstream pipes occurred in the simulation of this event. This resulted in the truncated shape of the computed hydrograph, as surcharged flows were first stored in manholes and later released (Figure 4-19). The shape of the rising limb before surcharge occurred, however, closely resembled that observed.

Storm of May 10, 1973

This simulation also produced surcharging resulting in the truncated shape of the outflow hydrograph (Figure 4-20). The rising limb of the computed hydrograph is advanced by about 10-15 minutes compared to the observed.

4.4.3 *Brucewood, North York*

Recent stormflow measurements were available for the 48 acre Brucewood area (Figure 4-21), and flow simulations were conducted using the default values for the various runoff parameters. All the recorded data were for relatively small storms.

Storm of May 14, 1974

Figure 4-22 shows the results of the simulation for this event, which produced one of the highest recorded peaks. The computed peak flow was some 18% lower than that recorded, and a general lag of 5-10 minutes was observed between the measured and computed hydrographs. The shape of the computed hydrograph agreed well with the observed, including a slow initial build-up and first peak before the major flow event. The volume of runoff computed was approximately 16% less than that recorded.

Storm of August 11, 1975

Figure 4-23 shows the results of simulation for this small rainfall event. The computed and measured peak flows were within 8% of one another, as were the runoff volumes. The shape of the computed hydrograph was quite similar to that recorded; however, the falling limb was delayed resulting in a broader hydrograph.

Storm of August 29, 1975

A very close agreement between measured and computed peak flows is evident in Figure 4-24. The initial peaks of this event were also well represented by the SWMM, although a more rapid flow recession, following peak flows was observed in the measurements. The total computed volume was approximately 10% greater than that recorded.

Storm of November 20, 1974

In this simulation (Figure 4-25), the computed peak flow was 3 cfs or 33% less than the recorded value. The first peak recorded was larger and better defined than the computed peak. The computed volume of flow was some 25% greater than recorded.

4.5 COMPARISON OF MODELS

4.5.1 *Runoff Flow Simulation*

The two largest storms used in the SWMM flow simulations for the Bannatyne district (June 19, September 5, 1971), were selected as the basis for a comparison of the quantity simulation routines of the SWMM, WREM and the Dorsch HVM. The RUNOFF models in the WREM and SWMM are similar. Consequently hydrographs from sub-watersheds were compared only for the SWMM and Dorsch HVM models. Both models were applied using the fine discretization shown in Figure 4-8. The peak flows generated in each of the 41 subcatchments by the SWMM and HVM Dorsch models are indicated on Figure 4-26 and Figure 4-27, for the storms of June 19 and September 5 respectively. For the former, it appears that the peak flows simulated by the Dorsch HVM are similar, but slightly higher than their SWMM equivalents. However, for the second storm the results are more evenly distributed about the 45 degree line. Flows draining from individual subcatchments were not recorded in the Bannatyne sampling program and consequently an absolute comparison of the models with measurements was not possible. A previous comparison of SWMM RUNOFF and Dorsch HVM with measurements carried on for the 10 acre area, Oakdale, Illinois, indicated that differences were not significant from a practical viewpoint. The results of the present comparison confirms that for small watersheds peak flows simulated by the two runoff models are very close. Both models use the kinematic wave approximation for overland flow and pipe routing effects for small areas are not considered significant.

4.5.2 *Transport Flow Simulation*

The following procedure was adhered to in order to obtain an objective comparison of the TRANSPORT flow simulation routines of the SWMM, Dorsch HVM and WRE models:

(a) The Bannatyne combined sewer district was discretized and the watershed and sewer system data were reduced according to the Dorsch HVM procedure.

(b) The inlet hydrographs to the sewer network were computed using the Dorsch HVM routine.

(c) The TRANSPORT routines of the three models were used to route the flow through the system, using the HVM inlet hydrographs as input.

(d) The procedure was carried out for two storm events - June 19, which was of low intensity and long duration, and September 5, which was of medium intensity and short duration.

The results of these comparative simulations are presented in Figures 4-28 and 4-29. For the first storm there is little difference between the simulations, which all agree well with measurements. For the second storm, the system was under surcharge.

The SWMM hydrograph shows the typical truncation effect due to the storage and later release of excess runoff. The WRE and Dorsch models reproduce the shape of the recorded hydrograph fairly well.

The models used were uncalibrated. However, it appears that with limited calibration the WRE and HVM models would accurately reproduce the measured hydrograph, whereas in the second, surcharged situation the SWMM would not.

Limitations of the SWMM as compared to WRE and Dorsch HVM become apparent at high flows when backwater effects or surcharge are important. The original SWMM was mainly a planning model developed for the study of pollution abatement methods. Comparatively rare events such as 7-10 year storms which are important for the design of relief sewers are not necessarily the critical events for the design of pollution abatement facilities (storage and/or treatment). Therefore, the discrepancies discussed above do not necessarily affect the use of SWMM as a planning tool.

For analysis of the existing systems for rare storm events or for final design applications, consideration of backwater and surcharge may result in savings in the capital cost of new works and the SWMM TRANSPORT routine should not be used. In this situation, input hydrographs generated using the SWMM can be routed by the Dorsch HVM or WRE TRANSPORT models, with superior results. The present work indicates that both more sophisticated models are operative and useful in practical applications. The non proprietary WRE Transport model is currently being incorporated by the University of Florida as an optional routine in the SWMM.

4.5.3 *RUNOFF and TRANSPORT Flow Simulation*

The limitations of the SWMM TRANSPORT routine in simulating flow under surcharged conditions have been discussed. In this section a global comparison of the Dorsch HVM and the SWMM is presented. The storms of May 10 and September 23, 1973 for the West Toronto area were used for these simulations. The May 10 storm caused surcharging, whereas the storm of September 23rd did not. The results of the simulations of each of the models are shown in Figures 4-30 and 4-31.

The Dorsch HVM simulations are based upon a very detailed discretization of the 2330 acre catchment that employed approximately 2000 conduits. The SWMM simulation, on the other hand, was based upon a discretization

employing 45 subcatchments and 71 conduit sections. These simulations were conducted with uncalibrated models. The default values in the SWMM were employed, and the Dorsch HVM model was run with standard values employed in their overall sewer system study for the City of Toronto. The measured flows, as discussed in Chapter 3, include approximations for diverted flows, and should not be used for absolute model comparisons.

For the storm of September 23, (Figure 4-30) the SWMM model produced a slightly higher peak flow and larger volume than did the Dorsch HVM, although the shape of the two computed hydrographs are similar and resemble that of the measured one. No significant surcharging occurred in the SWMM computations.

For the storm of May 10, (Figure 4-31) however, the SWMM computed hydrographs show the truncation effect due to stored surcharge water being released later on, while the Dorsch HVM hydrograph shows a much closer resemblance to the shape and magnitude of the measured hydrograph. The Dorsch simulation is uncalibrated, and could be made to represent the measured hydrograph very closely with minor calibration.

These results further indicate that the SWMM can produce a response under unsurcharged conditions which is not significatnly different from that of a more sophisticated model using a far greater amount of detail in the computations. Under surcharged conditions, the Dorsch model gives a superior simulation to that of the SWMM.

4.6 SIMULATION COSTS

Simulation costs for various models have been discussed in recent studies [1, 10]. Meaningful comparisons between models are clouded by differences in computers used and the rate schedules for on-line and off-line equipment. Other factors which affect the cost of a simulation are the amount

of printout, the extent of surcharged flow simulations, and the number of time steps employed. The following table summarizes typical run costs for different jobs, on the basis of the numerous runs done under the terms of this study. The SWMM simulations were performed on an IBM 370 model 168 computer at a basic rate of $640. per hour. The WRE model runs were on a CDC 6600 computer at a rate of $1,080. per hour. The Dorsch HVM simulations were conducted by Dorsch, and the computer costs reported represent amounts for computer time only.

TABLE 4.2

TYPICAL SIMULATION COSTS

	Description of Computer Run	No. of Sub-catchments	No. of Manholes & Conduits	No. of Time Steps	Computer Processing time (Min.)	Cost of Run
1)	SWMM RUNOFF + TRANSPORT, Quantity	45	159	40	1.153	$ 13.11
2)	SWMM RUNOFF + TRANSPORT, Quantity & Quality	27	82	80	.924	$ 9.57
3)	SWMM RUNOFF + TRANSPORT, Quantity & Quality	41	130	80	1.285	$ 19.21
4)	SWMM RUNOFF + TRANSPORT, Quantity & Quality	41	130	80	1.336	$ 19.67
5)	SWMM RUNOFF + TRANSPORT + Treatment Quantity & Quality	41	130	80	1.172	$ 23.18
6)	SWMM RUNOFF + TRANSPORT + Treatment + Receiving	41	130	80	2.826	$ 63.84
7)	WREM RUNOFF + TRANSPORT, Quantity	7	13	720	1.44	$ 26.12
8)	WREM RUNOFF + TRANSPORT, Quantity	29	85	1440	4.885	$ 87.88
9)	WREM RUNOFF + TRANSPORT, Quantity	128	300	560	6.700	$157.64
10)	Dorsch HVM Model, Quantity	41	126	-	-	$100.00
11)	Dorsch HVM Model, Quantity	2000	2000	-	-	$750.00

Most SWMM simulations involve the RUNOFF and TRANSPORT blocks only, although in future the TREATMENT and RECEIVING blocks will be increasingly applied. On the basis of the simulations done in this study, the following has been prepared as an approximate guide to the user in the estimation of the costs of running the SWMM:

For runs involving RUNOFF and TRANSPORT only:

Cost in Dollars = total no. of elements x no. of time steps x \$0.0025

For runs involving RUNOFF, TRANSPORT, STORAGE/TREATMENT and RECEIV:

Cost in Dollars = total no. of elements x no. of time steps x \$0.0050

These approximations are based upon the computer system utilized in this study, as discussed above. Several runs will give any user a feel for costs on his own computer.

4.7 CONCLUSIONS

(a) A sensitivity analysis has indicated the relative influence of the major parameters in both the RUNOFF and TRANSPORT blocks. This analysis has further indicated that the existing SWMM default values should be retained.

(b) The Dorsch HVM and SWMM generate similar subcatchment overland flow hydrographs for a given rainfall pattern. This result is expected since both models employ the Kinematic Wave approximation for overland flow routing.

(c) Under normal flow conditions, with no surcharging of the sewer system, the Dorsch HVM, WRE and SWMM TRANSPORT routines produce similar hydrographs for a given set of inlet hydrographs. The quasi-steady dynamic wave formulation of the SWMM is fairly simple. However, good results are produced for conduits in which there are no excessive backwater effects.

(d) In the case of drainage systems under surcharge and with backwater conditions the SWMM does not correctly reproduce the shape of the recorded hydrograph. The Dorsch HVM and WRE models utilize a pressurized flow routine for surcharged conditions, thereby reflecting the increased capacity of pipes under surcharge.

In addition , the solution of the dynamic wave equation over the entire sewer system at each time step allows for a proper simulation of the dynamic effect of the backwater condition.

REFERENCE - CHAPTER 4

1. Brandstetter, A., "Assessment of Mathematical Models for Storm and Combined Sewer Management", Preliminary report, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, 1975.

2. James F. MacLaren Ltd., "Review of Canadian Design Practice and Comparison of Urban Runoff", Canada-U.S. Agreement Task 3/4 Project 4, October 1975, Project No. 74-8-31, obtainable from Training and Technology Transfer Division (Water), Environmental Protection Service, Environment Canada, Ottawa.

3. Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc. Storm Water Management Model. U.S. Environmental Protection Agency Report 11024 DOC 07/71, 4 Volumes, October 1971.

4. Mevius, F., "Analysis of Urban Sewer Systems by Hydrograph - Volume Method", paper presented at the National Conference on Urban Engineering Terrain Problems, Montreal, May 1973.

5. Dorsch Consult Ltd., "Sewer System Analysis by the Hydrograph Volume Method", Dorsch Consult, Toronto, 1974.

6. Geiger, F.W., "Simulation of Urban Runoff Pollution", Dorsch Consult Inc., Munich, Germany. 1975.

7. Shubinski, R.P., and L.A. Roesner. Linked Process Routing Models. Paper presented at American Geophysical Union Annual Spring Meeting, Washington, D.C., April 1973.

8. Water Resources Engineers Inc., "Modifications to the EPA Storm Water Managment Model Documentation", Water Resources Engineers Inc., September 1975.

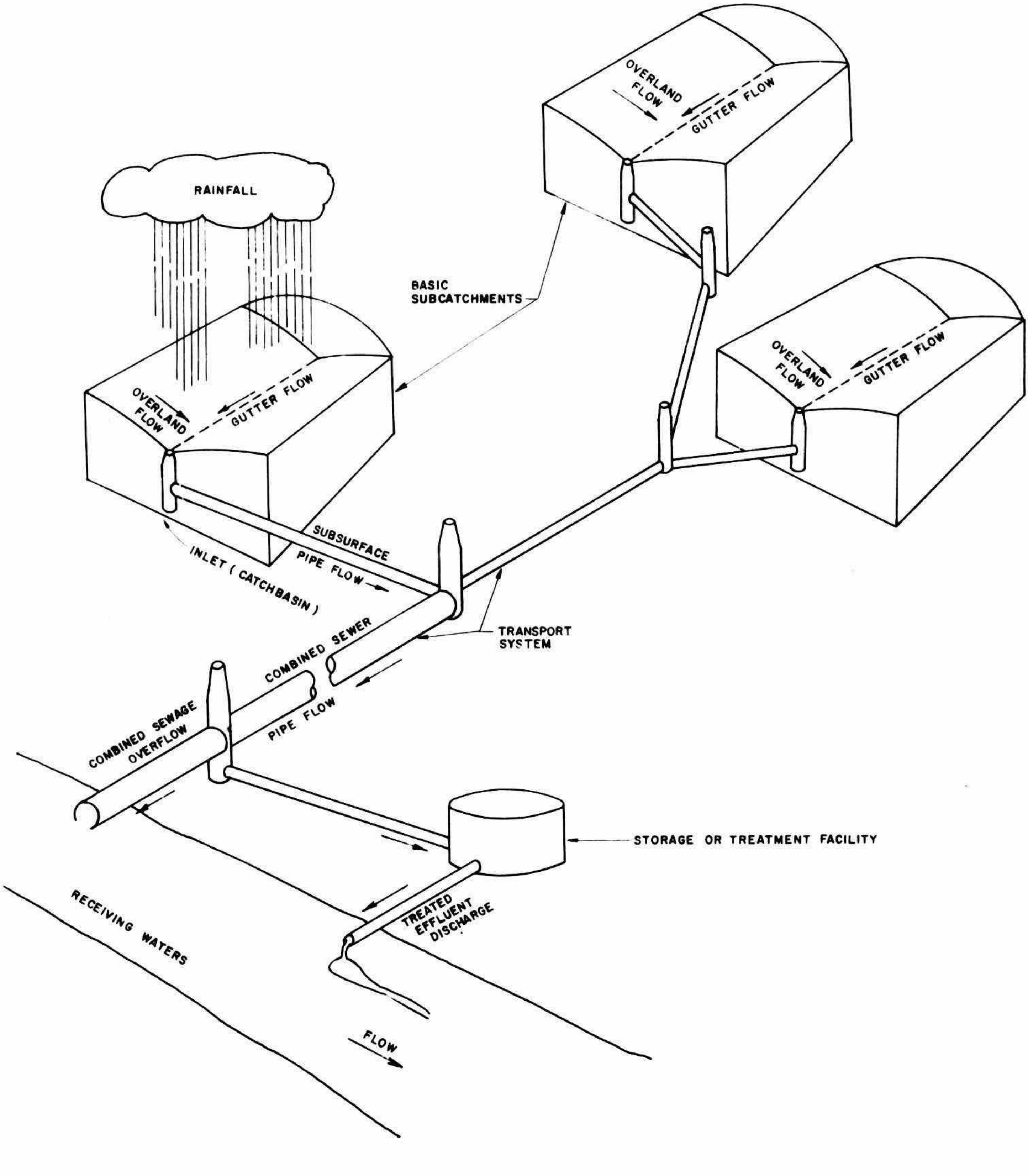
9. Graham, P.H., et al, "Estimation of Imperviousness and Specific Curb Length for Forecasting Stormwater Quality and Quantity", Journal of the Water Pollution Control Federation, Volume 46, April 1974.

10. Hodgson, J., "A Comparison of Three Runoff Simulation Models", Unpublished Paper, Vancouver Sewerage and Drainage District, Vancouver, B.C., 1974.

URBAN DRAINAGE SYSTEM

SCHEMATIC

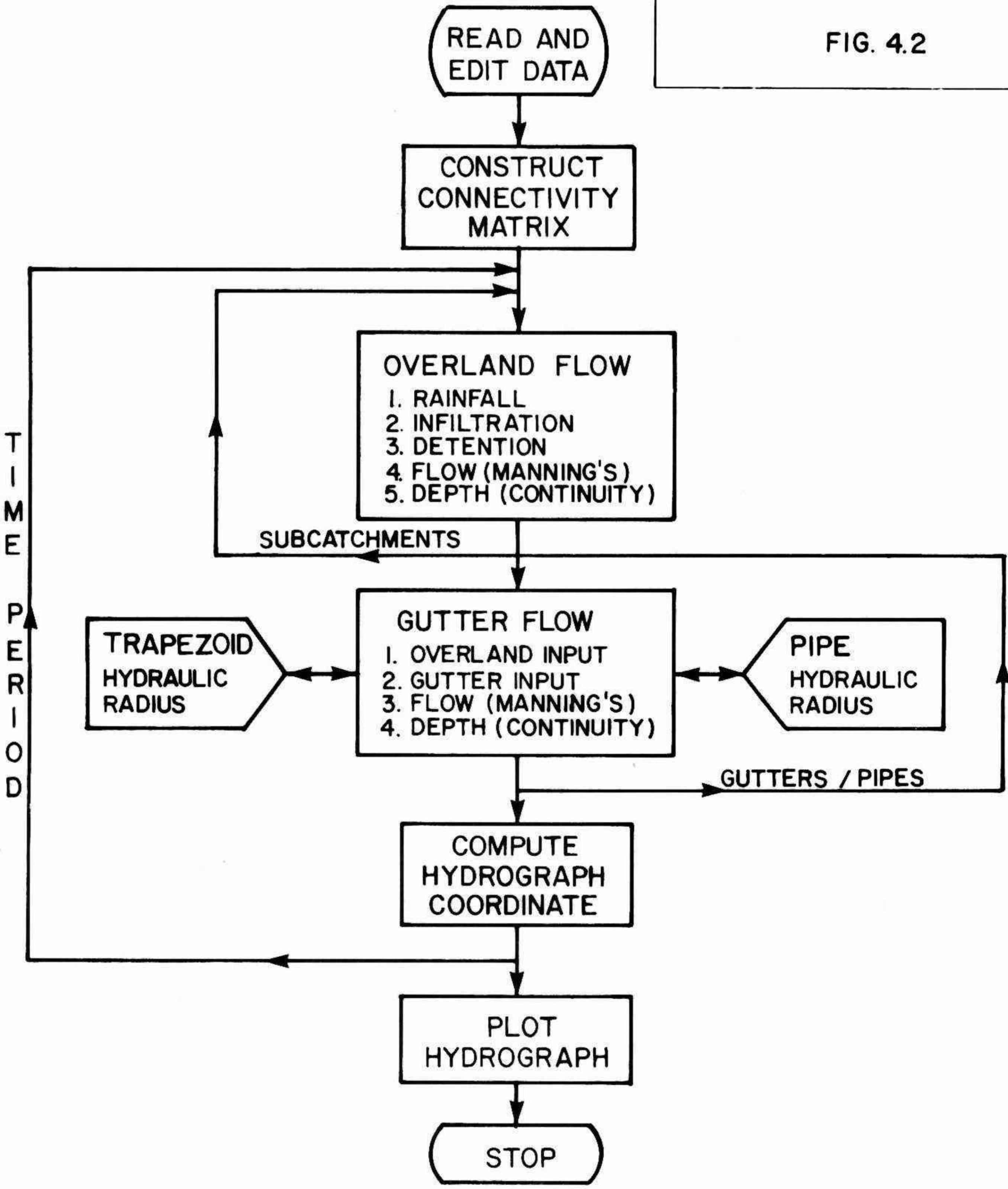
FIG. 4.1



FLOW CHART

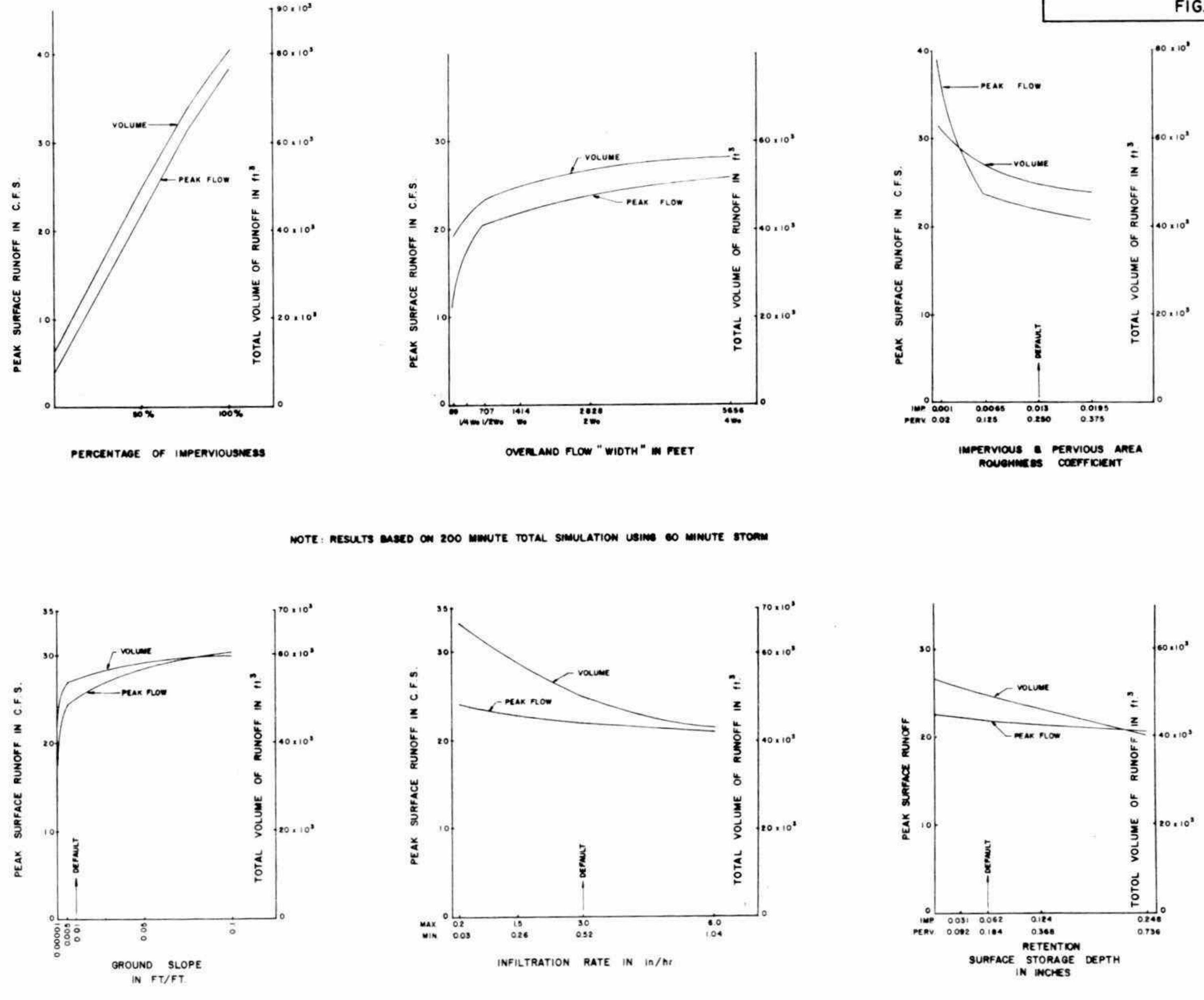
HYDROGRAPHIC COMPUTATION

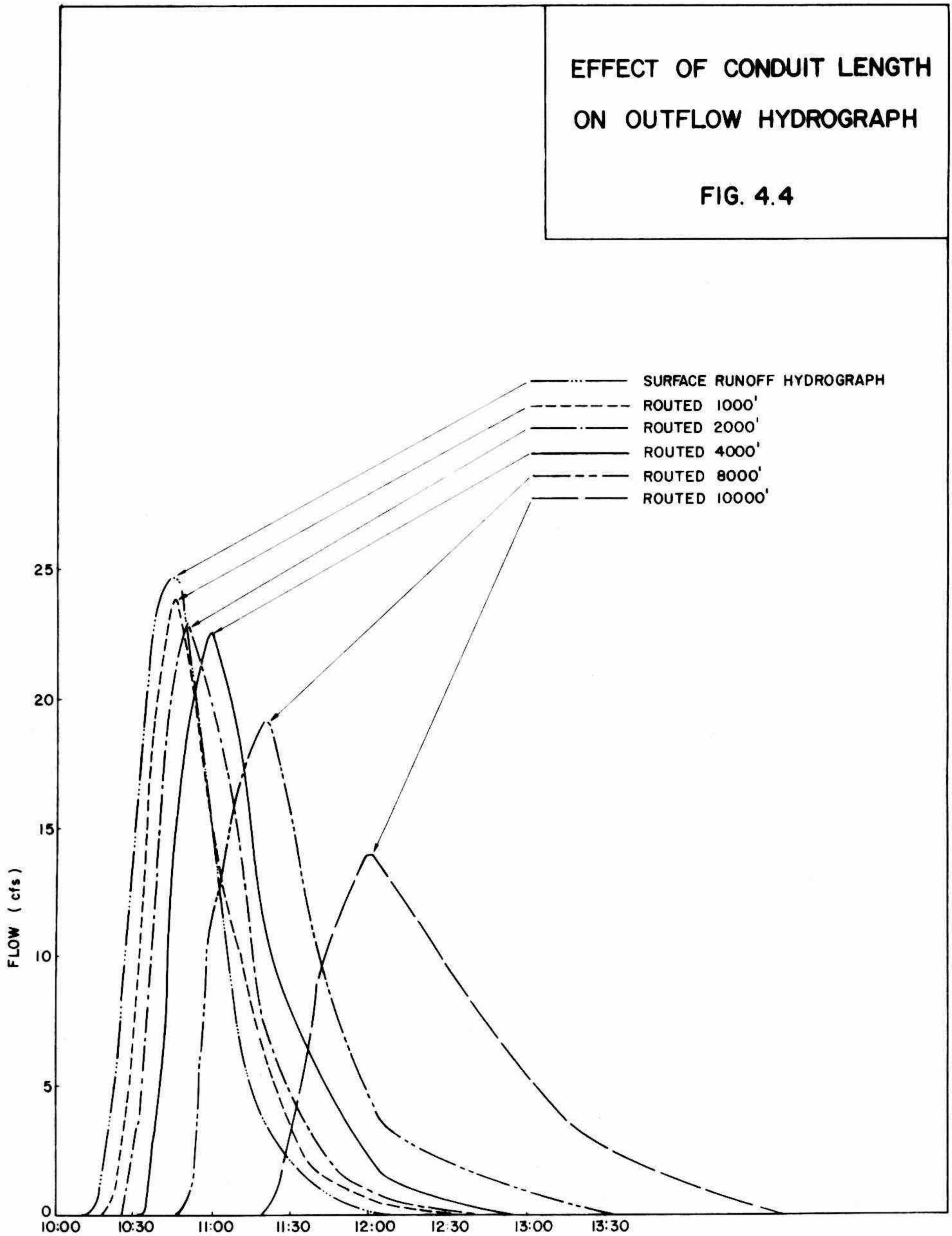
FIG. 4.2

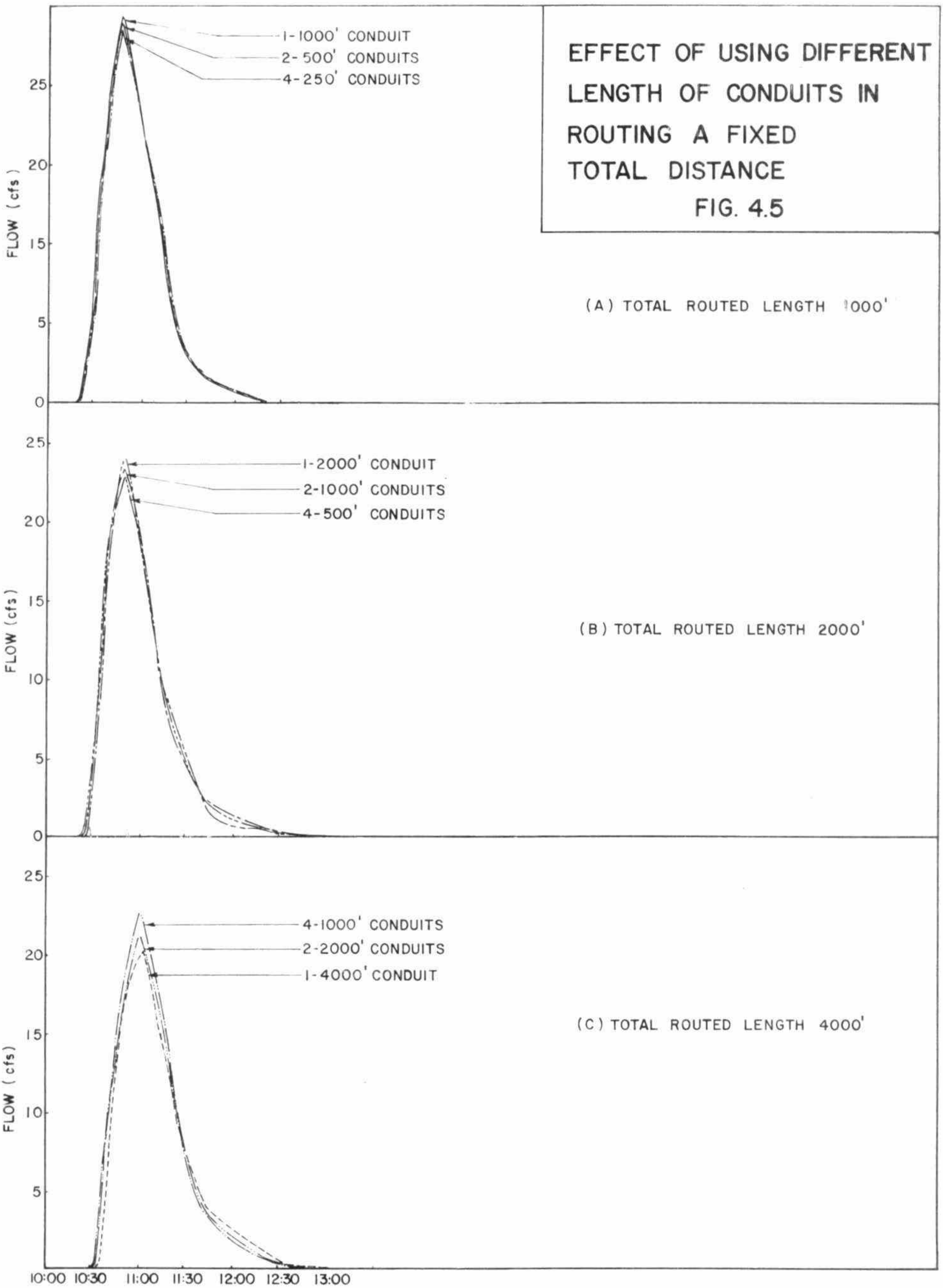


RUNOFF BLOCK SENSITIVITY RESULTS FROM II ACRE HYPOTHETICAL SUBCATCHMENT

FIG. 4.3




EFFECT OF CONDUIT LENGTH ON OUTFLOW HYDROGRAPH
FIG. 4.4
SURFACE RUNOFF HYDROGRAPH
ROUTED 1000'
ROUTED 2000'
ROUTED 4000'
ROUTED 8000'
ROUTED 10000'
FLOW (cfs)
0
5
10
15
20
25
10:00
10:30
11:00
11:30
12:00
12:30
13:00
13:30
TIME (hrs.)

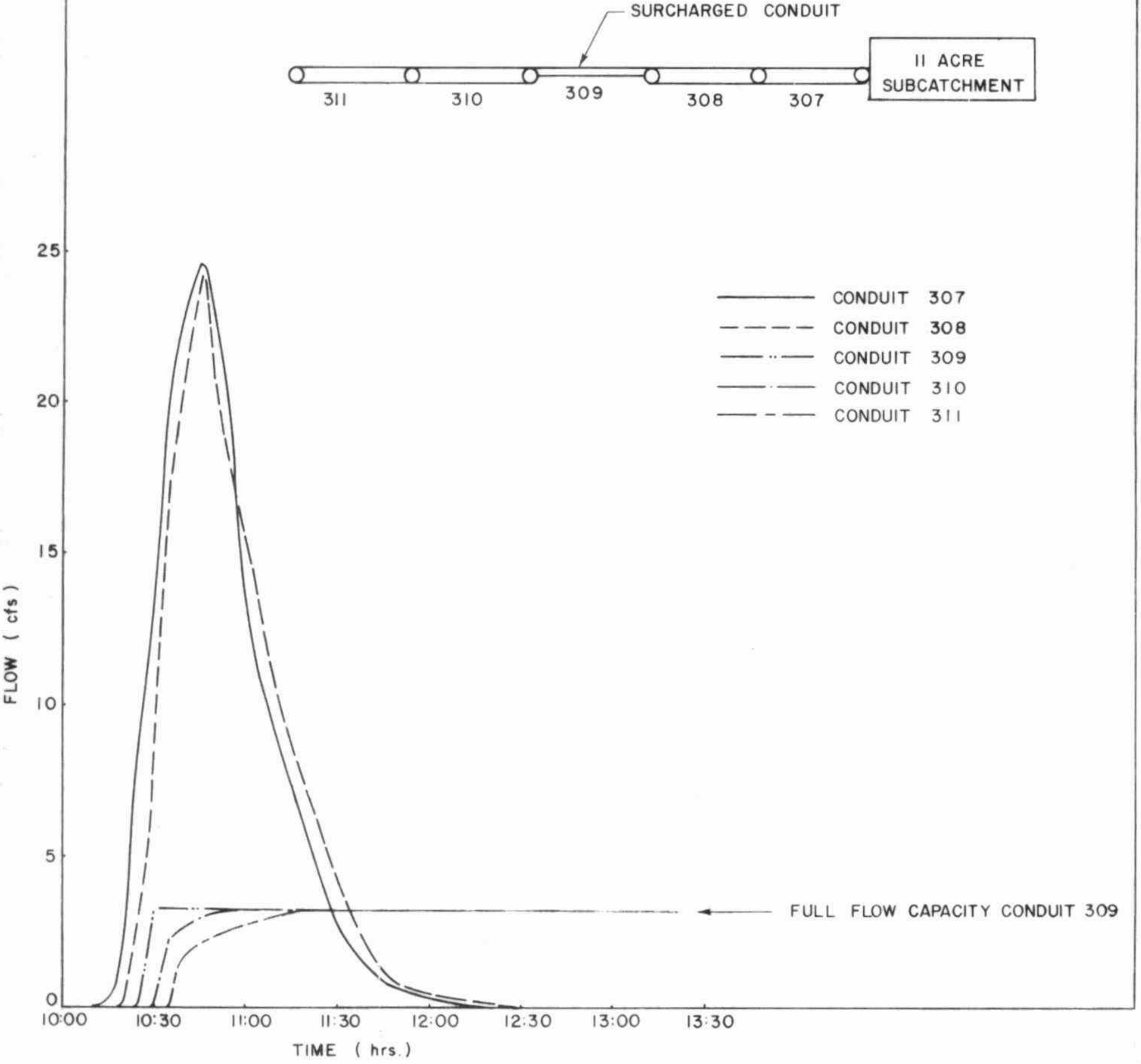


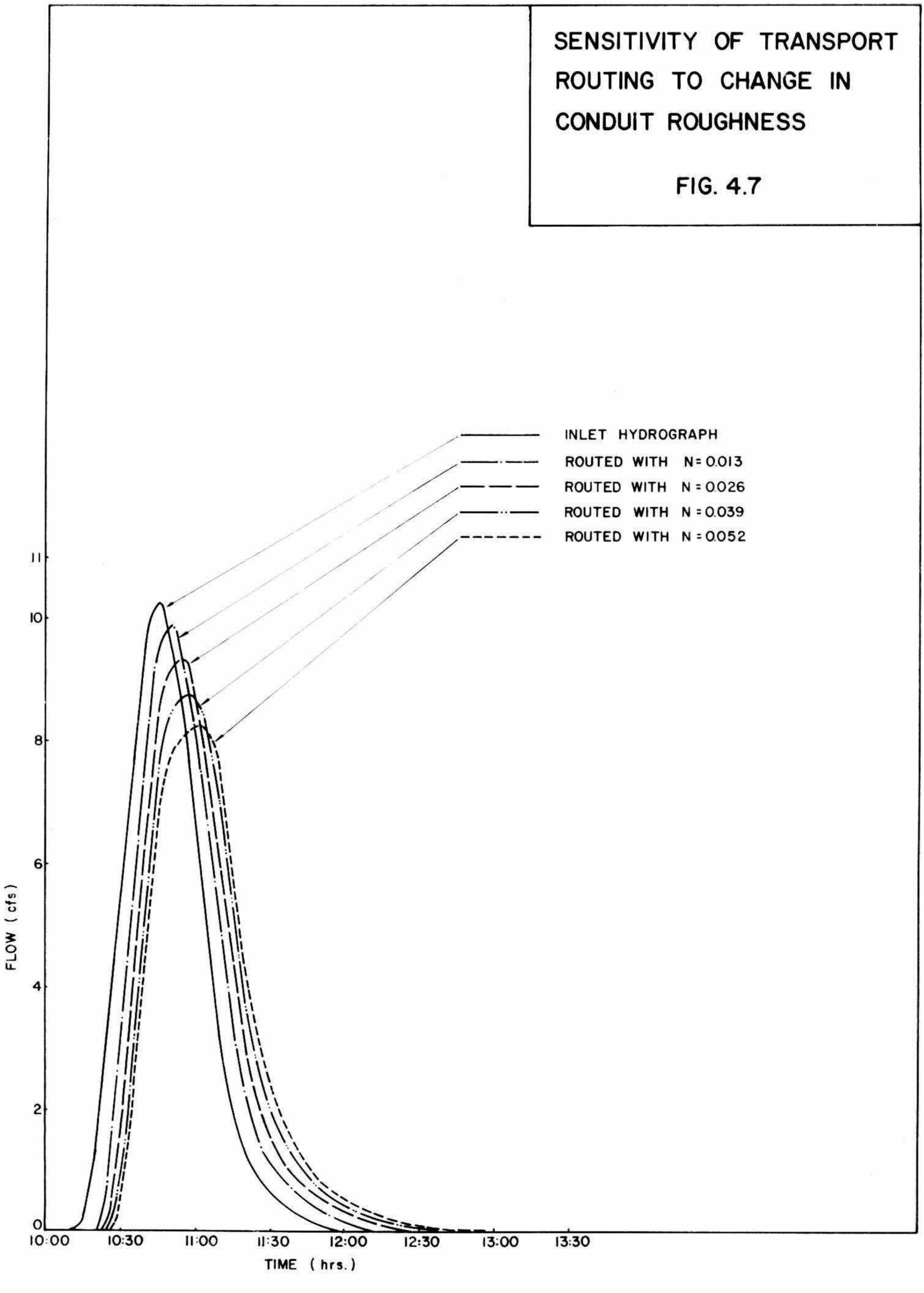
EFFECT OF USING DIFFERENT LENGTH OF CONDUITS IN ROUTING A FIXED TOTAL DISTANCE

FIG. 4.5

EFFECT OF SURCHARGE

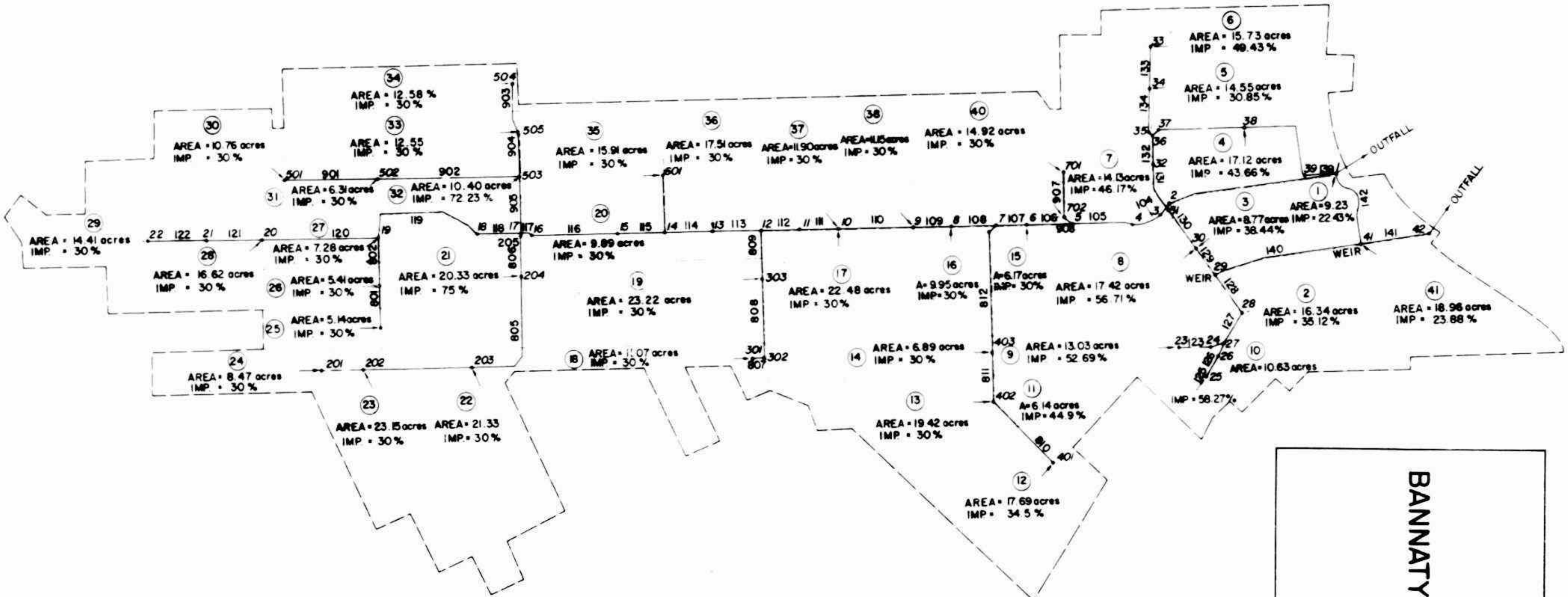
FIG. 4.6





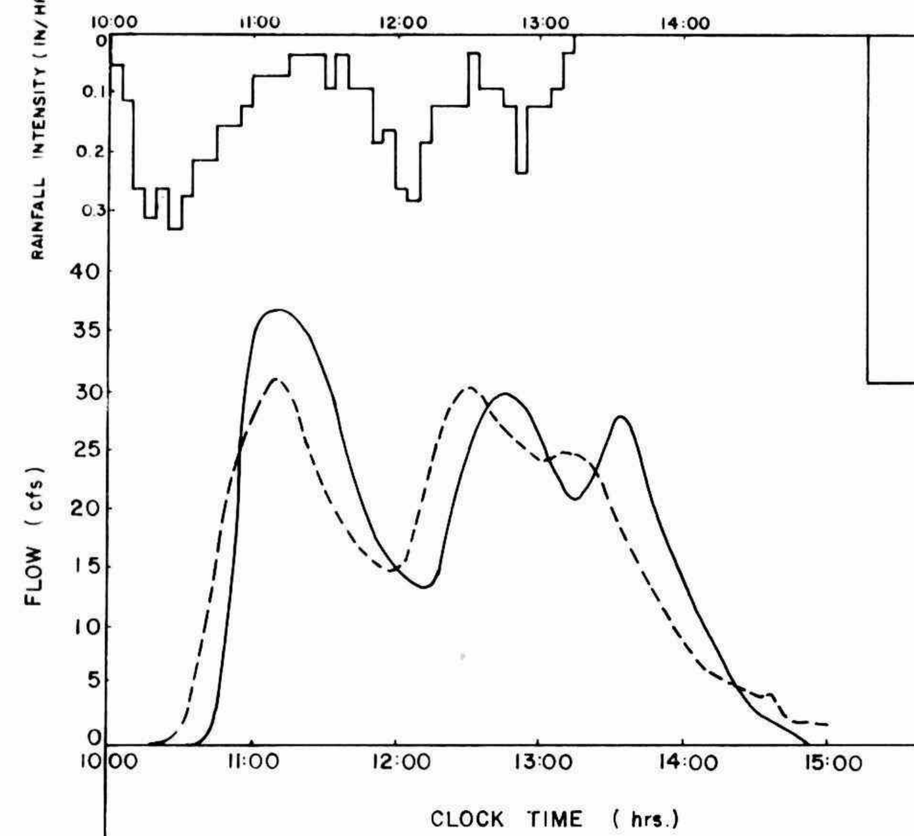
SENSITIVITY OF TRANSPORT ROUTING TO CHANGE IN CONDUIT ROUGHNESS

FIG. 4.7



BANNATYNE SEWER DISTRICT

FIG. 4.8

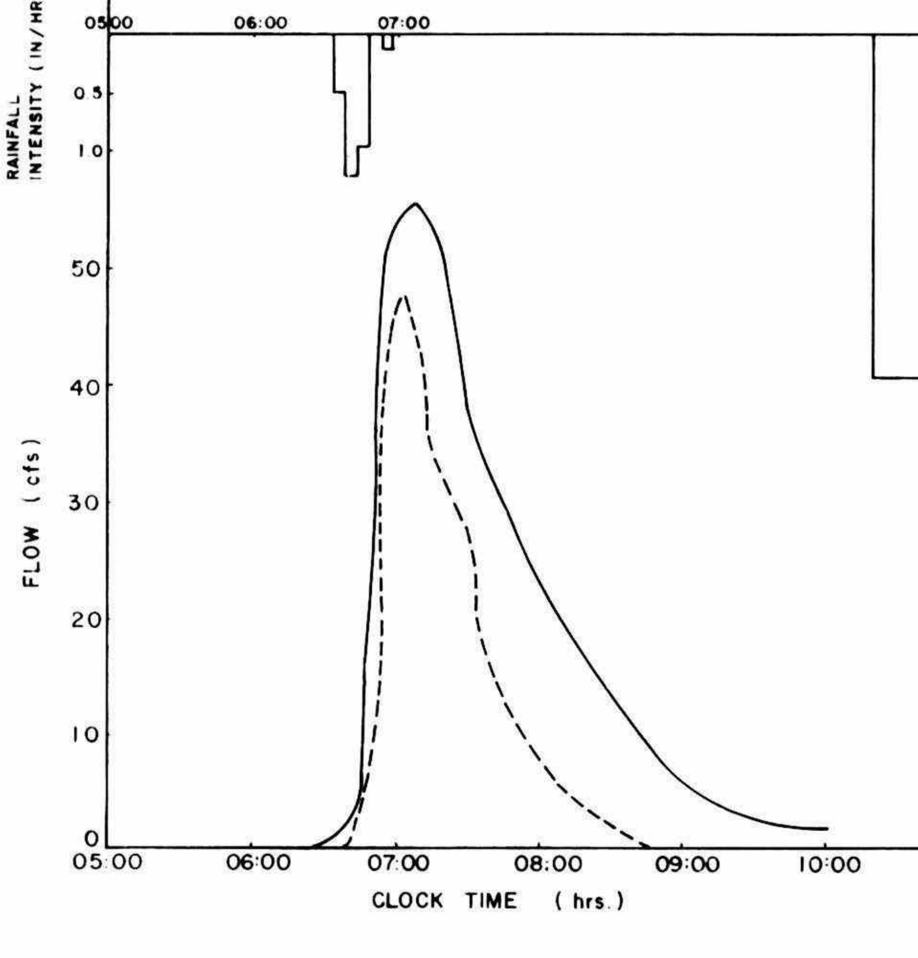


SWMM SIMULATION

BANNATYNE DISTRICT

STORM OF JUNE 19, 1971

FIG. 4.9

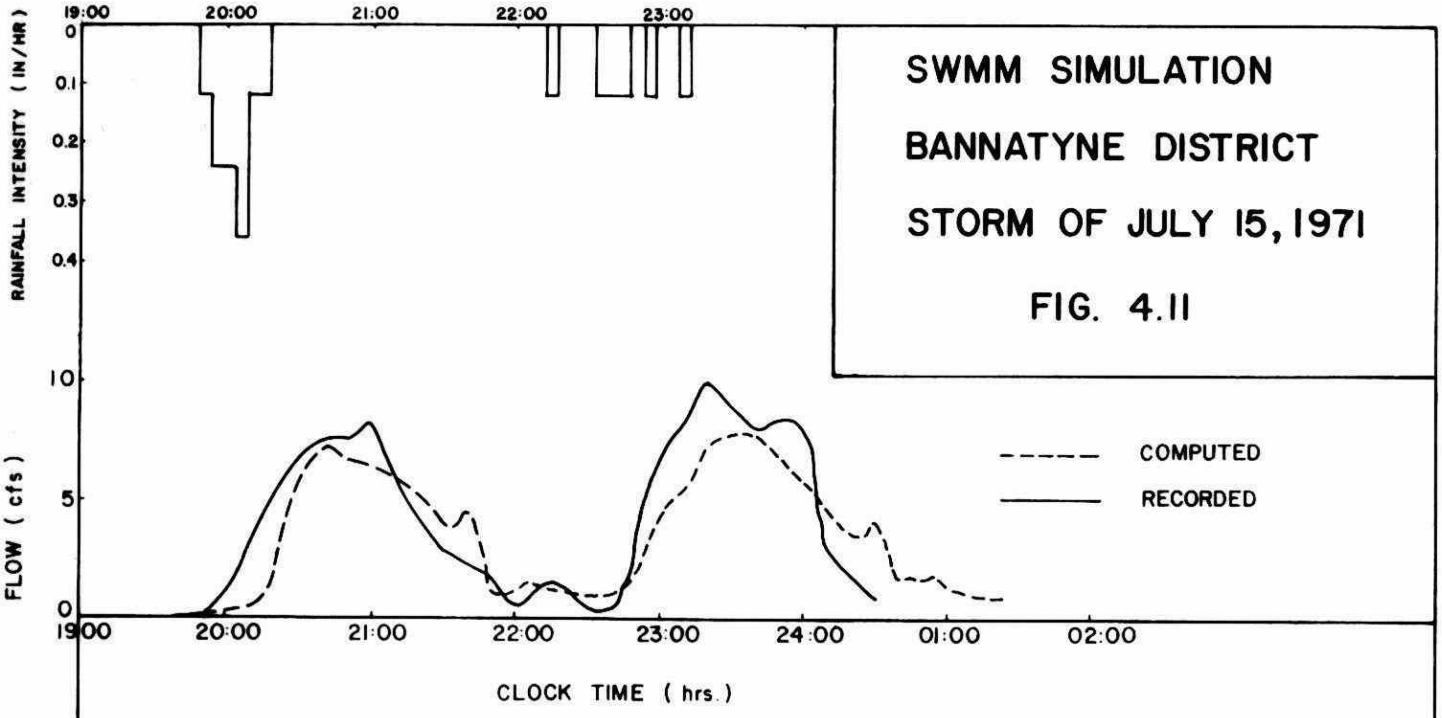


SWMM SIMULATION

BANNATYNE DISTRICT

STORM OF JULY 3, 1971

FIG. 4.10

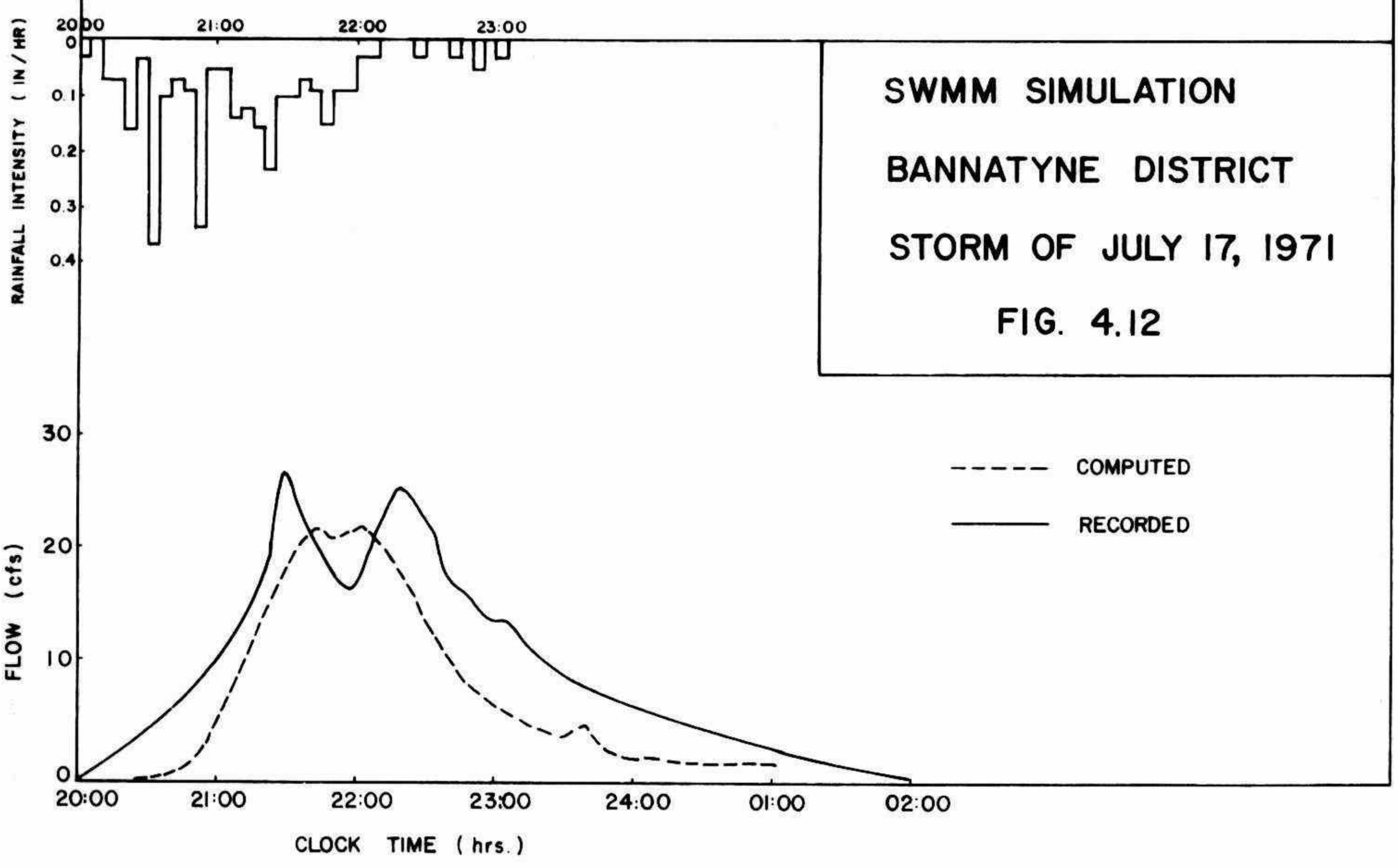


SWMM SIMULATION

BANNATYNE DISTRICT

STORM OF JULY 15, 1971

FIG. 4.11

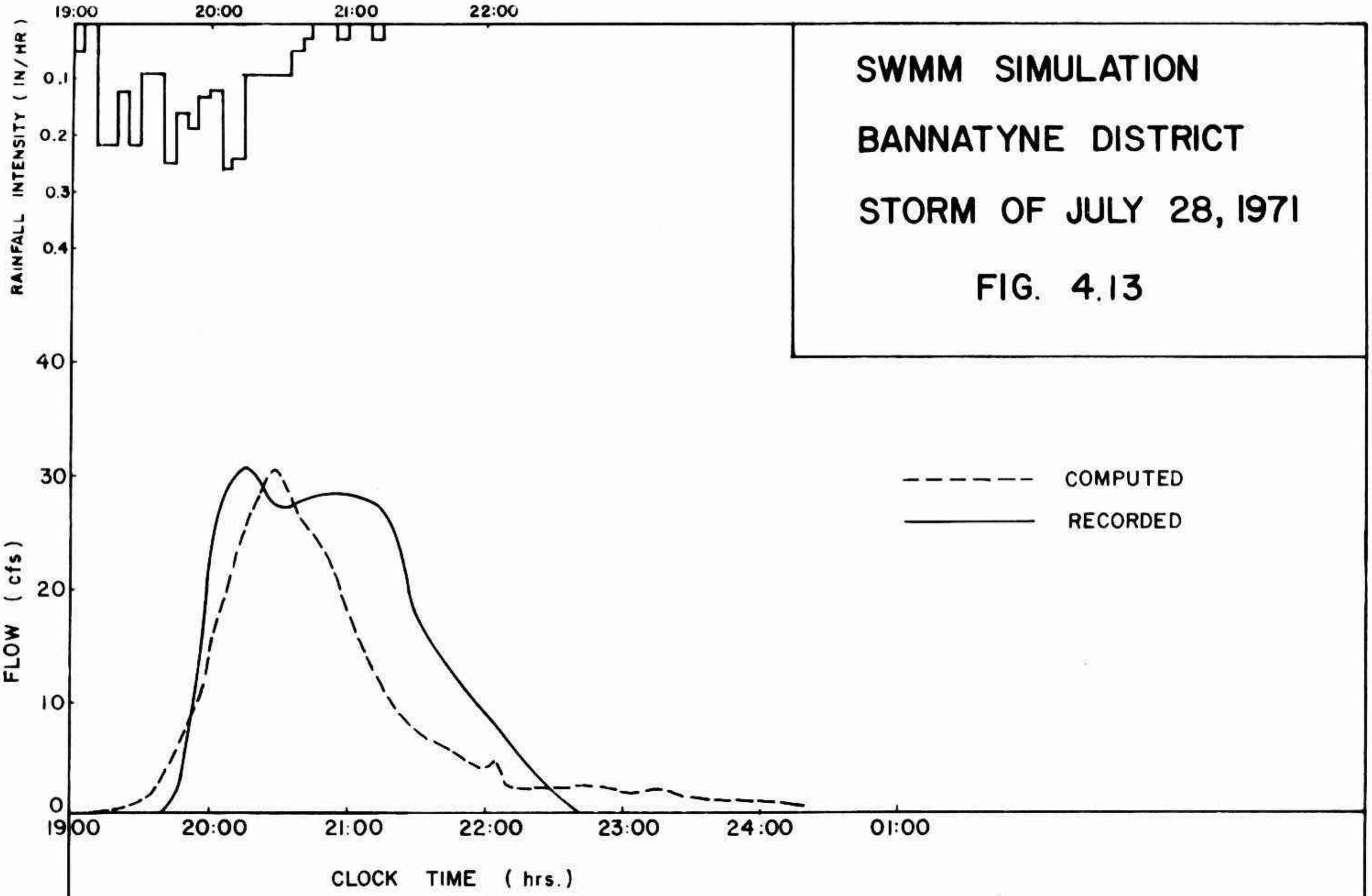


SWMM SIMULATION

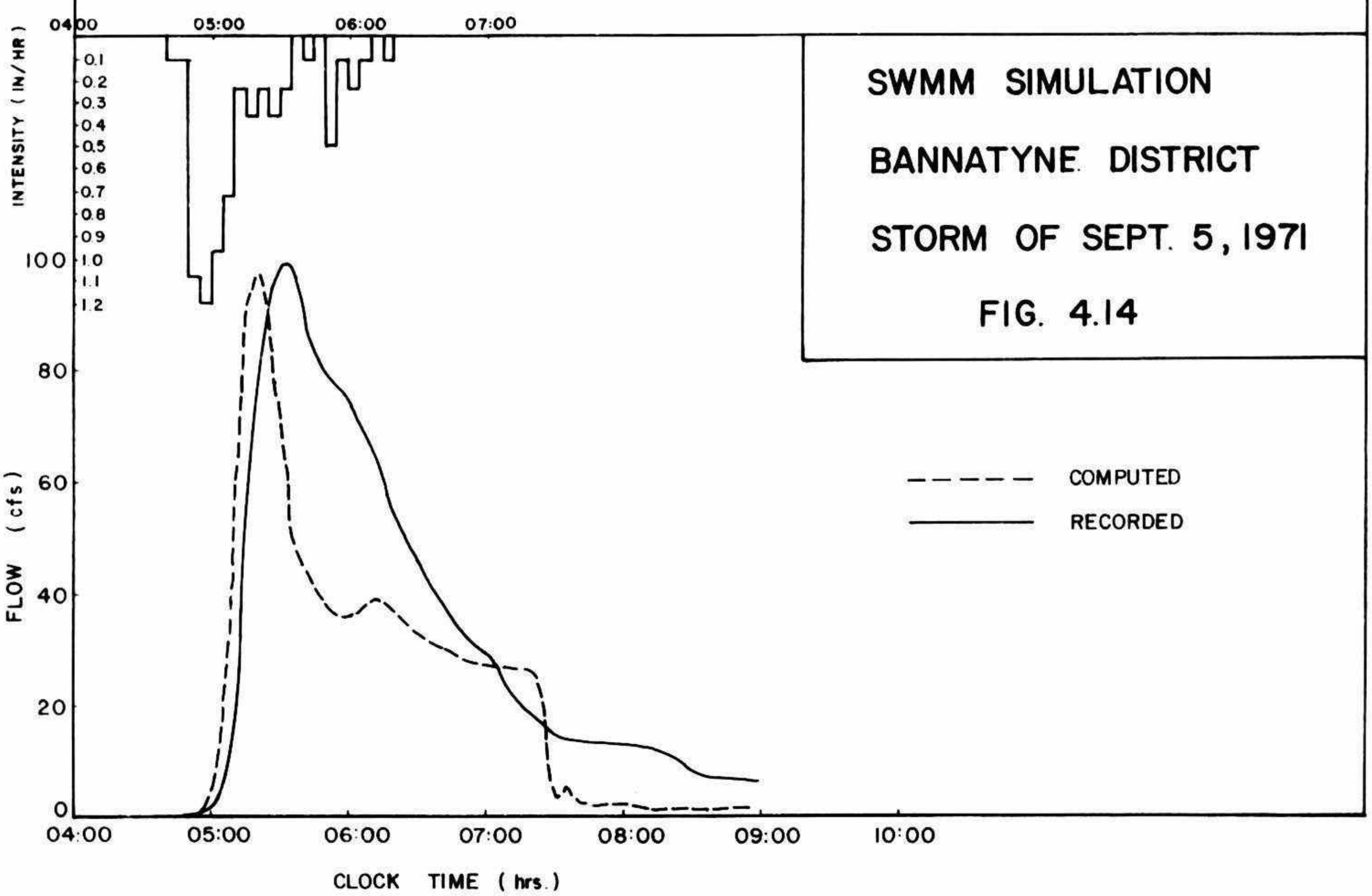
BANNATYNE DISTRICT

STORM OF JULY 17, 1971

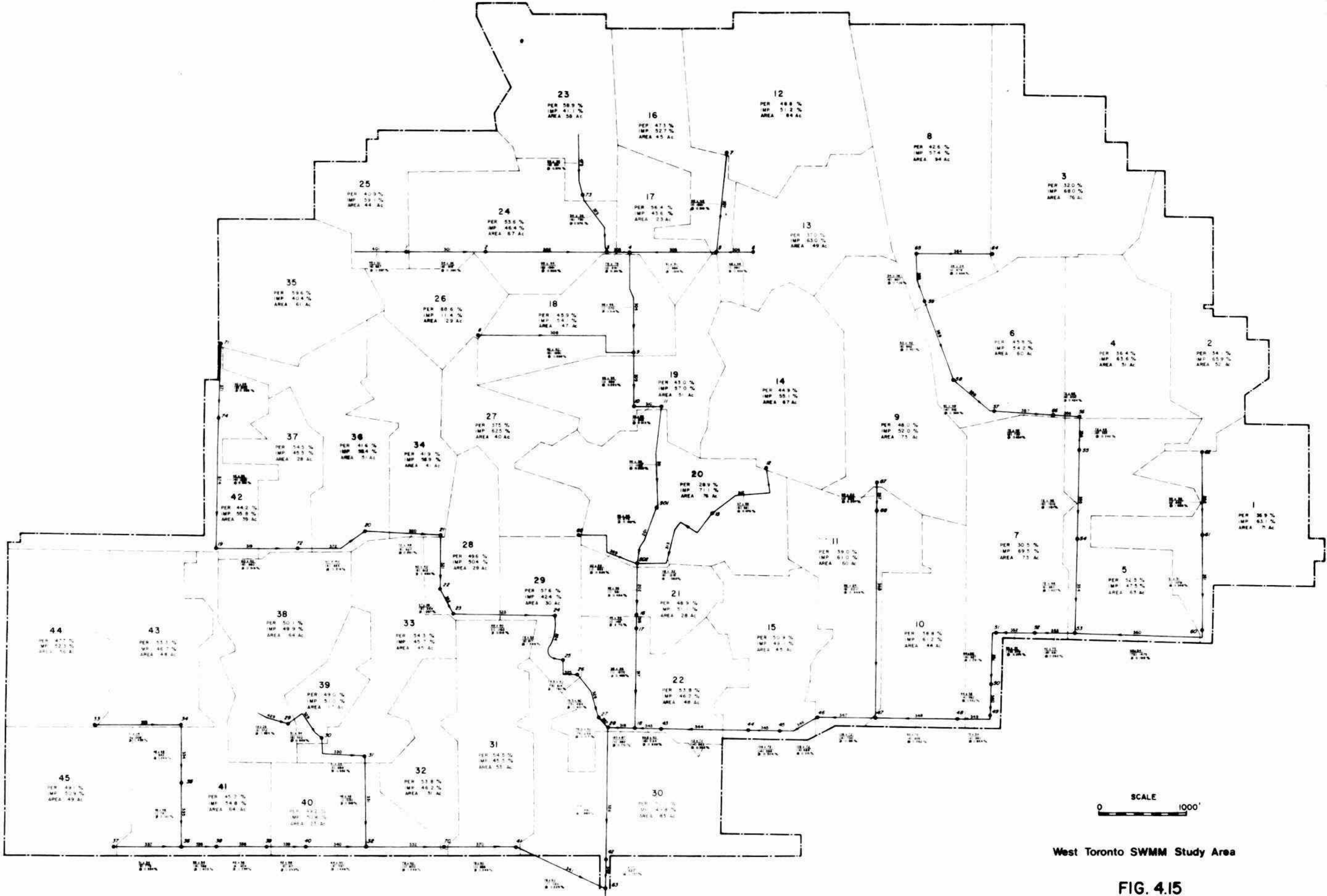
FIG. 4.12



SWMM SIMULATION
BANNATYNE DISTRICT
STORM OF JULY 28, 1971
FIG. 4.13

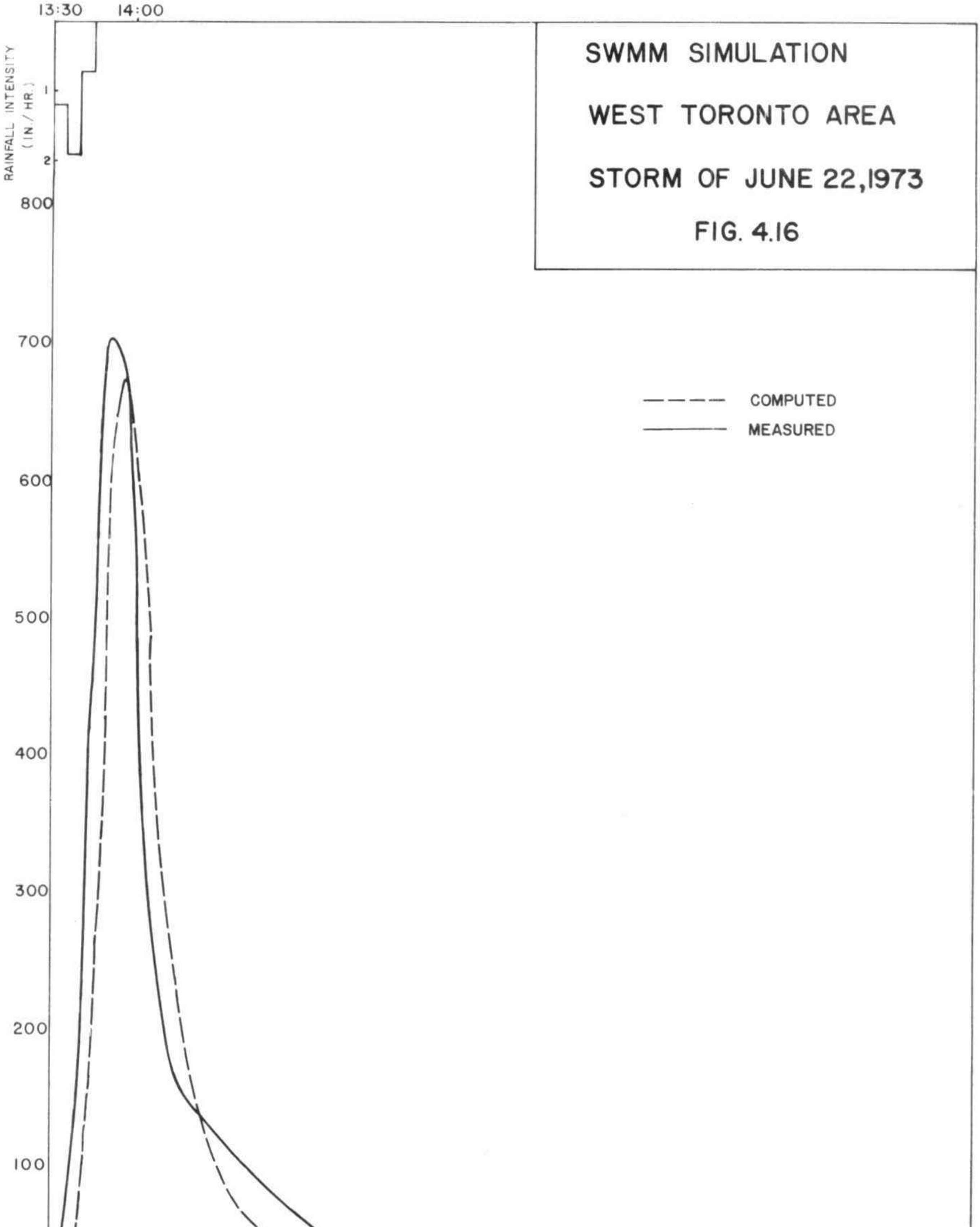


SWMM SIMULATION
BANNATYNE DISTRICT
STORM OF SEPT. 5, 1971
FIG. 4.14

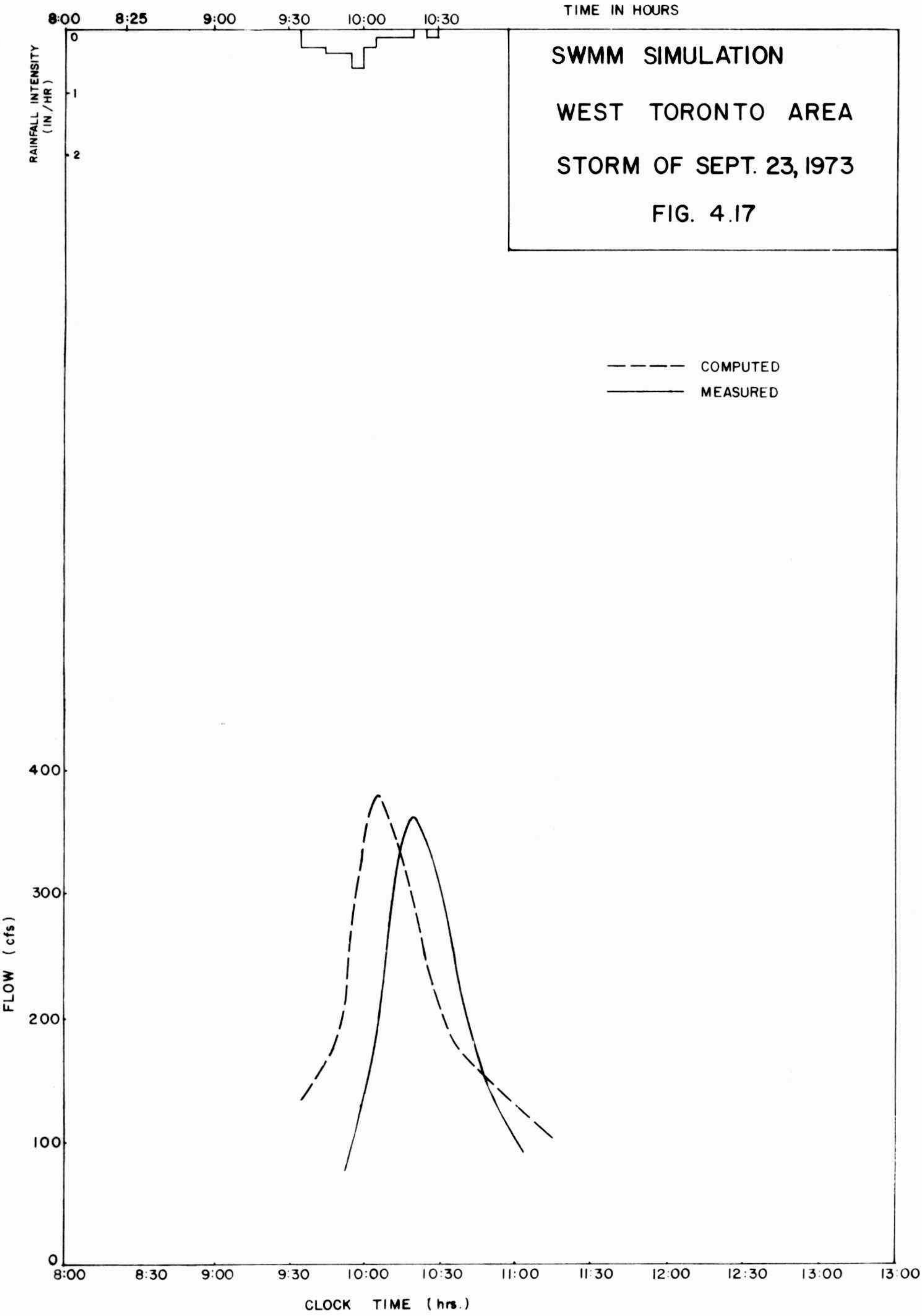


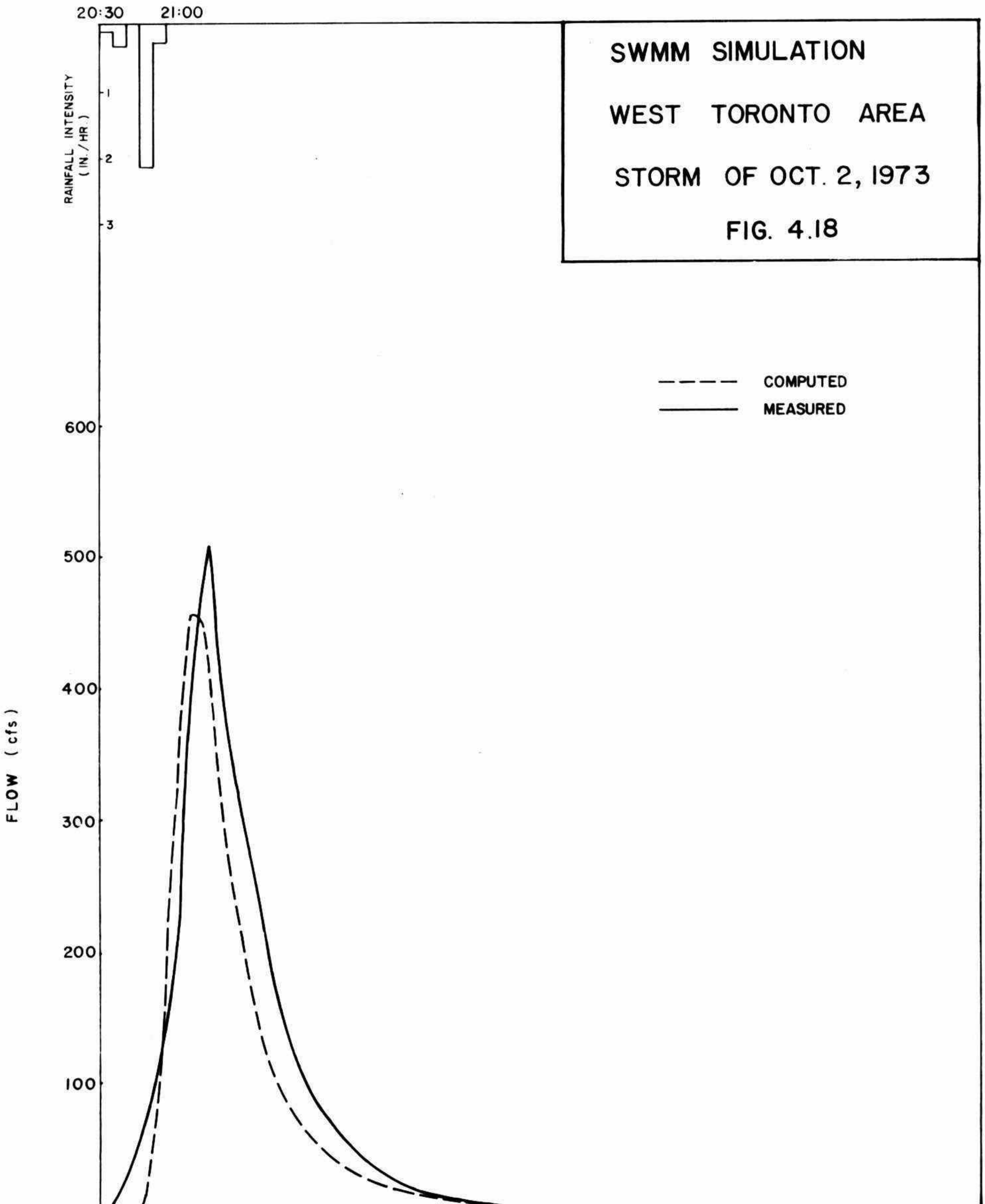
West Toronto SWMM Study Area

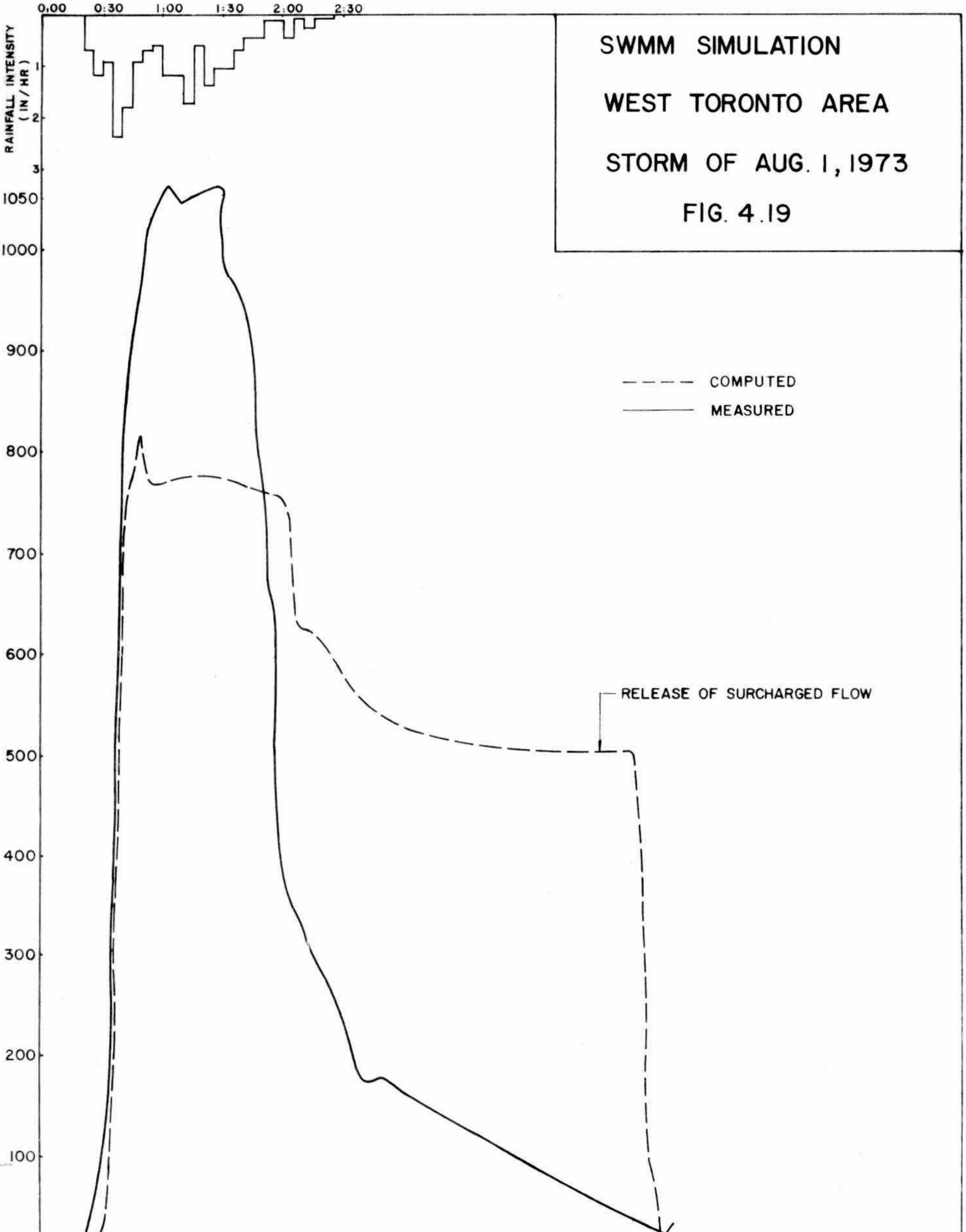
FIG. 4.15



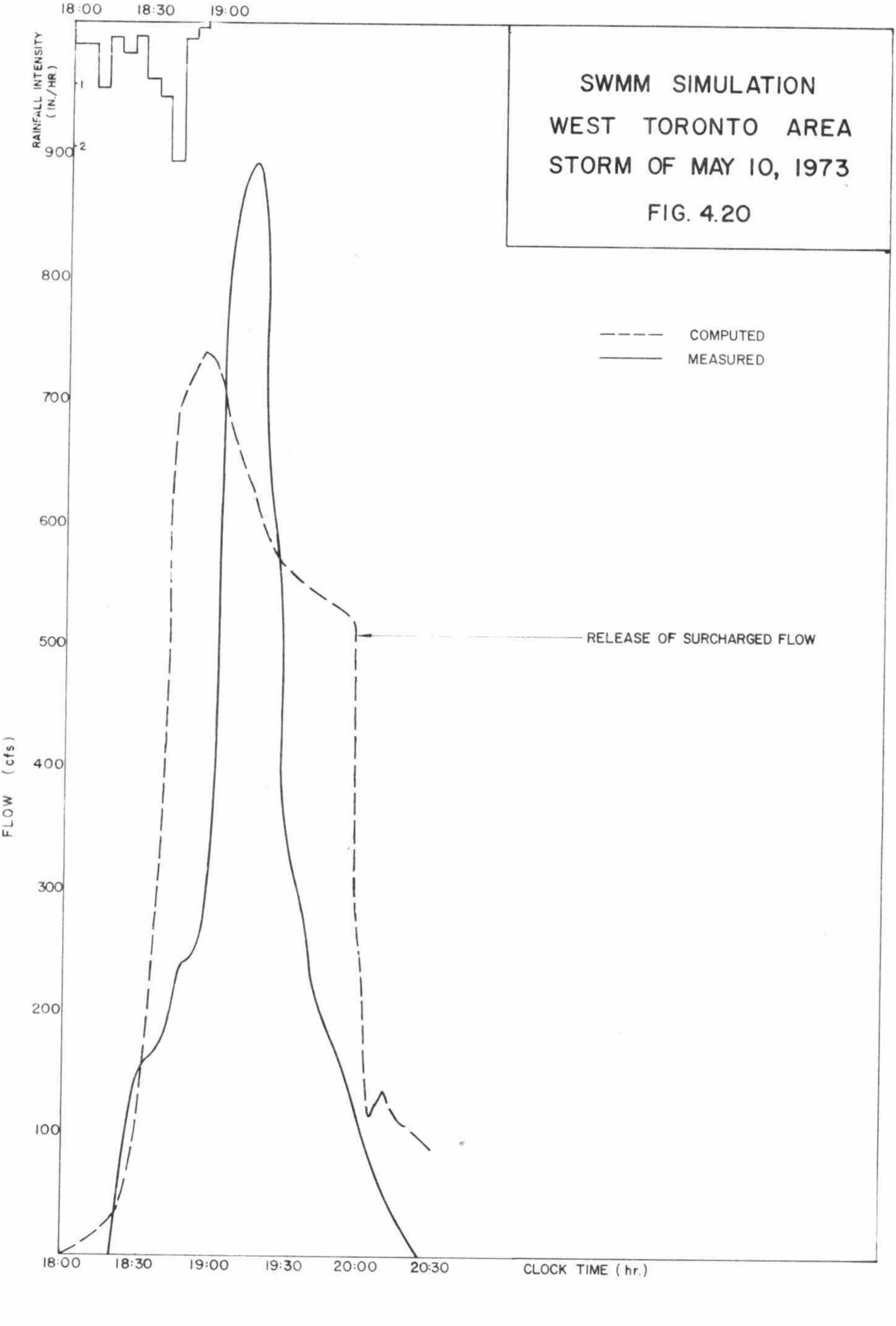
SWMM SIMULATION
WEST TORONTO AREA
STORM OF JUNE 22,1973
FIG. 4.16
COMPUTED
MEASURED
RAINFALL INTENSITY (IN./HR.)
1
2
13:30
14:00
800
700
600
500
400
300
200
100
0
FLOW (cfs)
13:30
14:00
14:30
15:00
15:30
16:00
16:30
CLOCK TIME (hrs.)


TIME IN HOURS
8:00
8:25
9:00
9:30
10:00
10:30
RAINFALL INTENSITY (IN / HR)
0
1
2
SWMM SIMULATION
WEST TORONTO AREA
STORM OF SEPT. 23, 1973
FIG. 4.17
COMPUTED
MEASURED
FLOW (cfs)
400
300
200
100
0
8:00
8:30
9:00
9:30
10:00
10:30
11:00
11:30
12:00
12:30
13:00
13:00
CLOCK TIME (hrs.)


SWMM SIMULATION
WEST TORONTO AREA
STORM OF OCT. 2, 1973
FIG. 4.18
COMPUTED
MEASURED
RAINFALL INTENSITY (IN./HR.)
1
2
3
20:30
21:00
FLOW (cfs)
0
100
200
300
400
500
600
20:30
21:00
21:30
22:00
22:30
23:00
23:30
24:00
0:30
1:00
CLOCK TIME (hrs.)



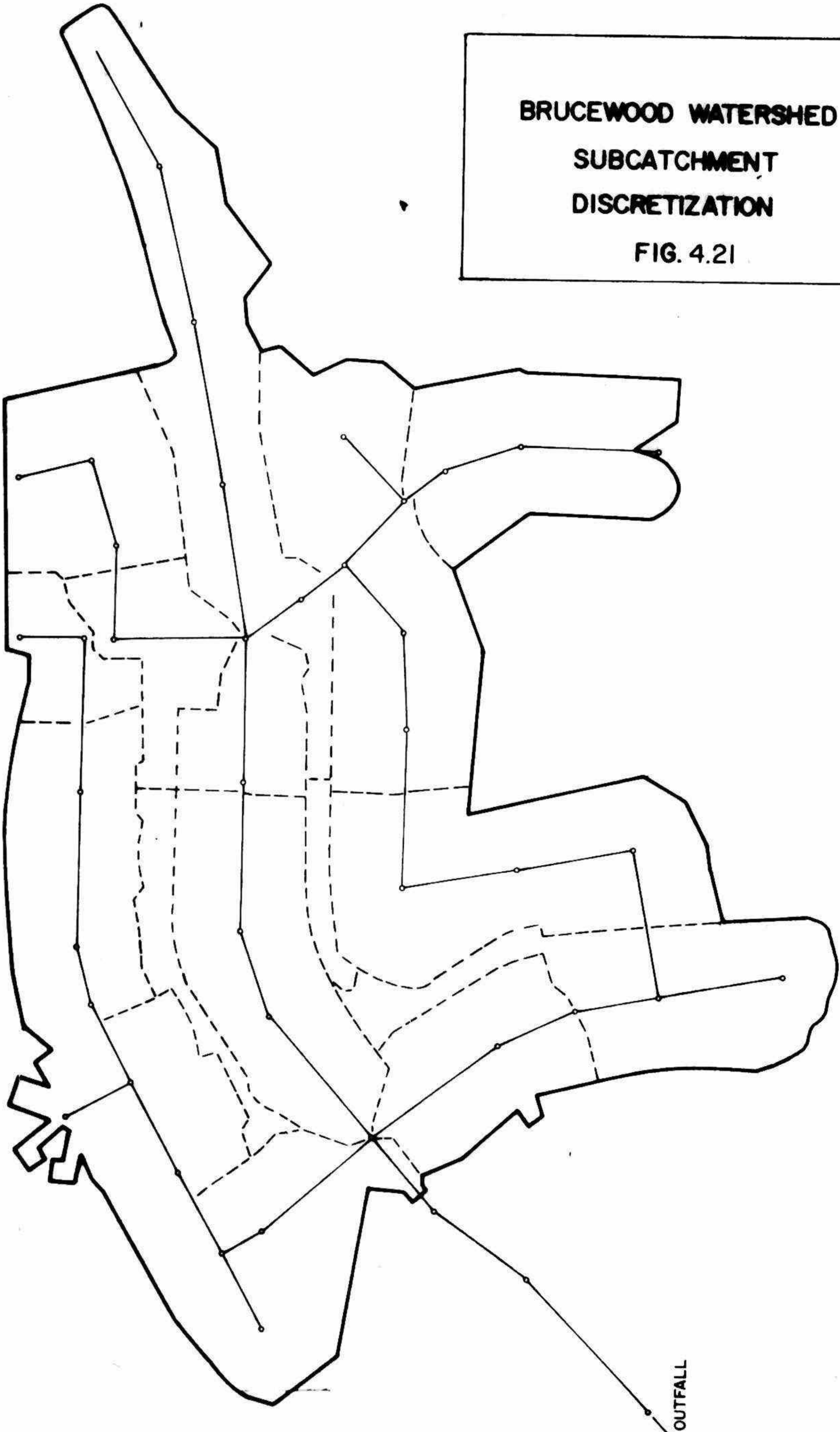
SWMM SIMULATION
WEST TORONTO AREA
STORM OF AUG. 1, 1973
FIG. 4.19
RAINFALL INTENSITY (IN / HR)
0:00 0:30 1:00 1:30 2:00 2:30
1
2
3
COMPUTED
MEASURED
RELEASE OF SURCHARGED FLOW
FLOW (cfs)
1050
1000
900
800
700
600
500
400
300
200
100
0
0:00 0:30 1:00 1:30 2:00 2:30 3:00 3:30 4:00 4:30 5:00 5:30
CLOCK TIME (hrs.)

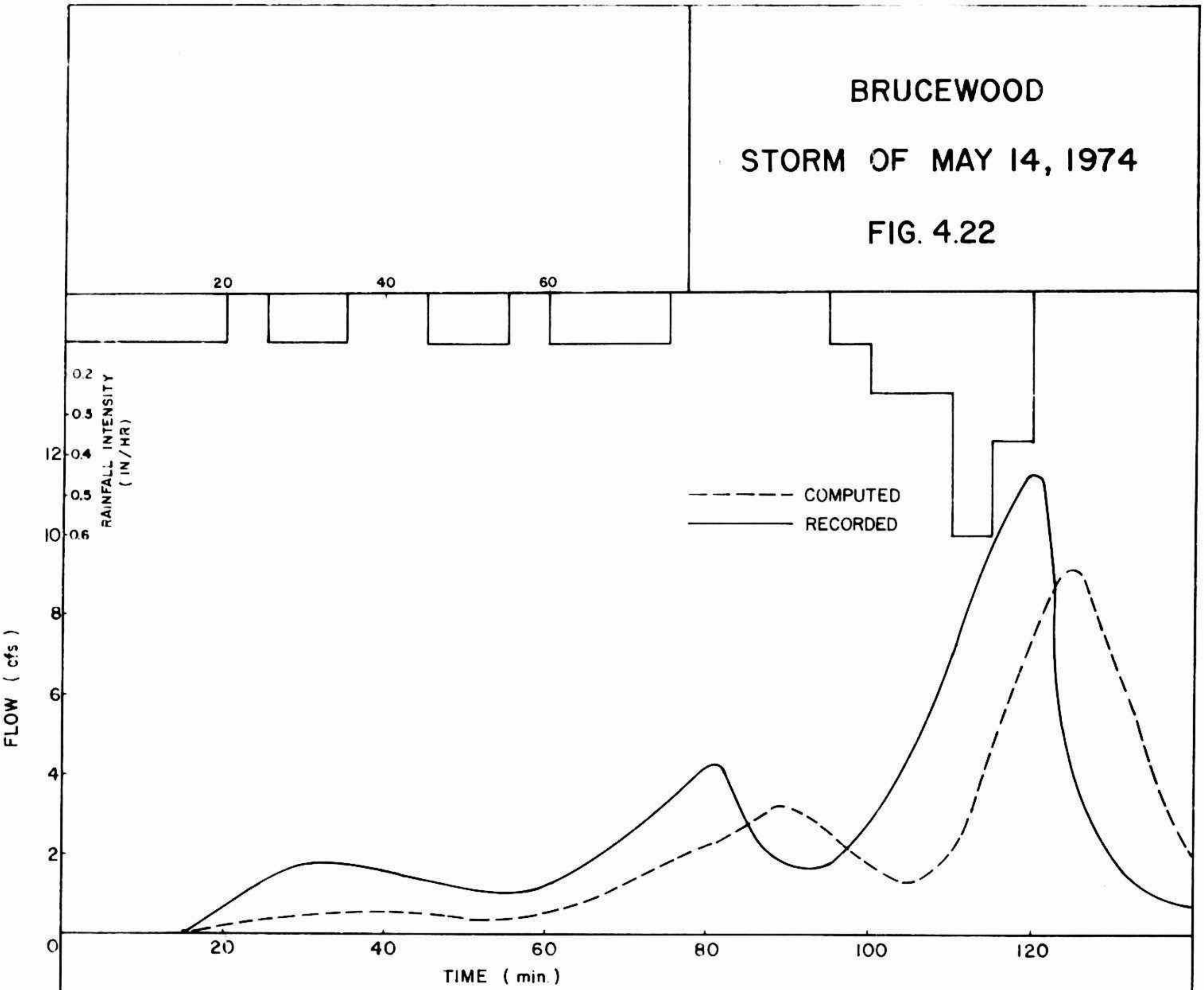


SWMM SIMULATION
WEST TORONTO AREA
STORM OF MAY 10, 1973
FIG. 4.20
COMPUTED
MEASURED
RELEASE OF SURCHARGED FLOW
RAINFALL INTENSITY (IN./HR.)
18:00
18:30
19:00
1
2
FLOW (cfs)
900
800
700
600
500
400
300
200
100
18:00
18:30
19:00
19:30
20:00
20:30
CLOCK TIME (hr.)

BRUCEWOOD WATERSHED SUBCATCHMENT DISCRETIZATION

FIG. 4.21

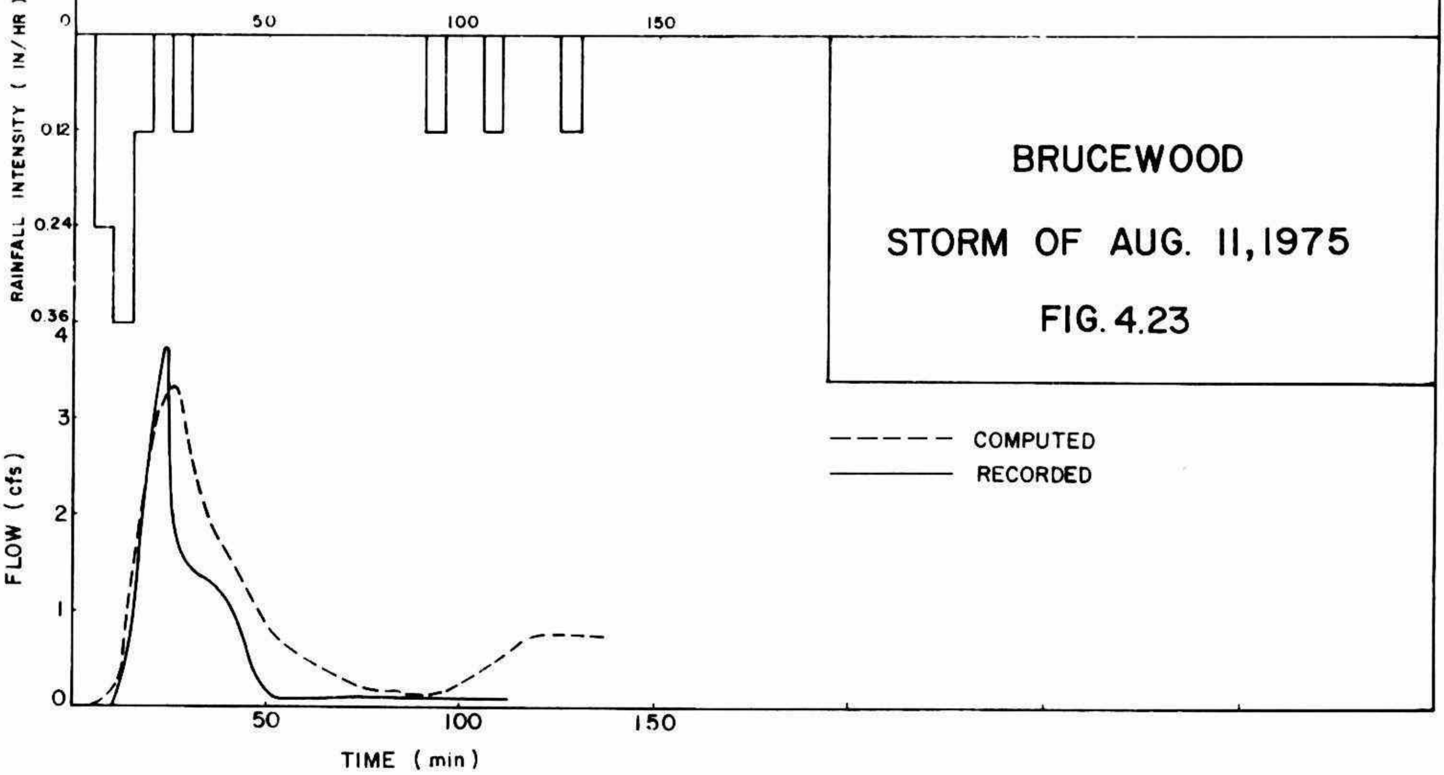




BRUCEWOOD

STORM OF MAY 14, 1974

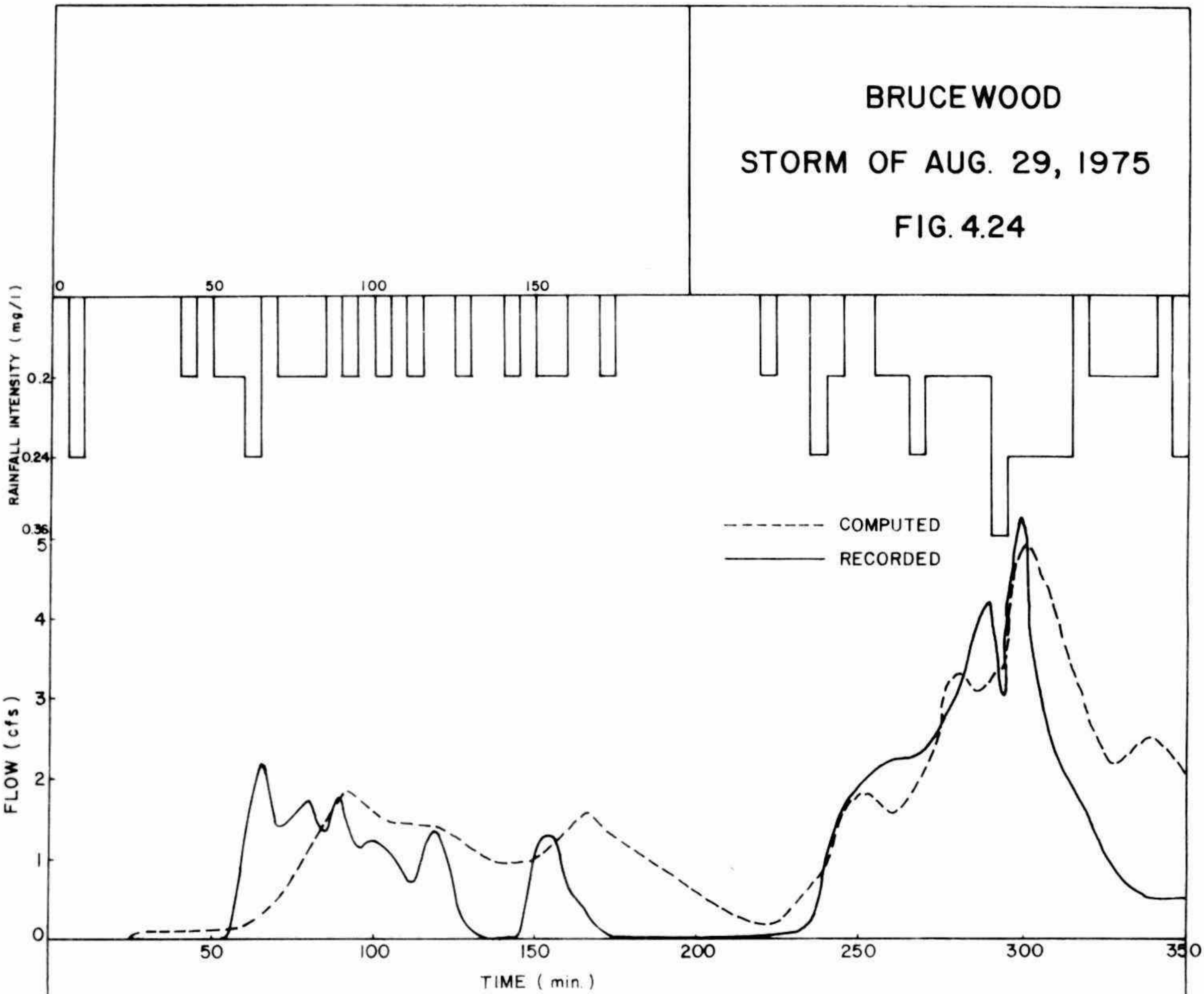
FIG. 4.22



BRUCEWOOD

STORM OF AUG. 11, 1975

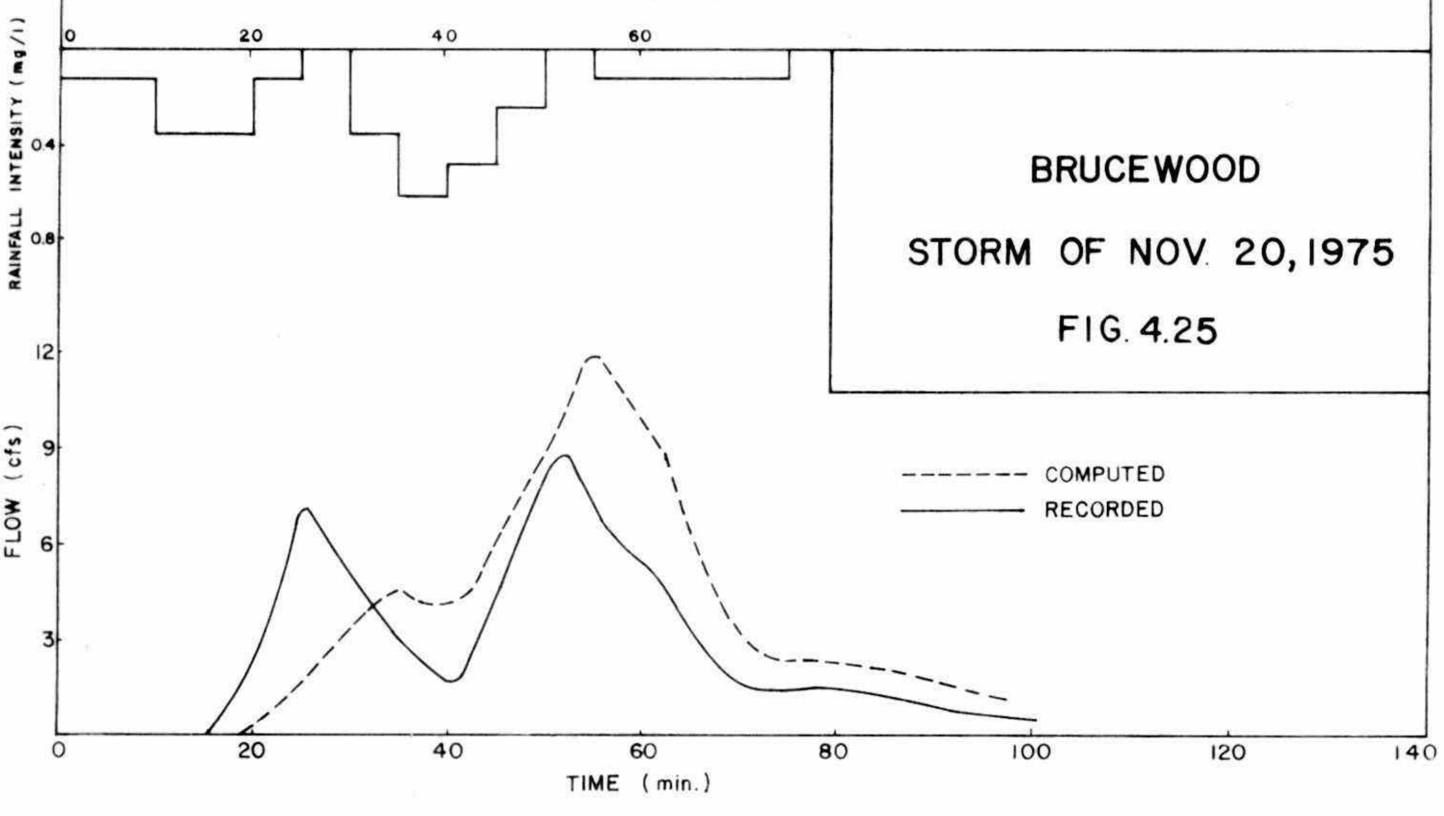
FIG. 4.23



BRUCEWOOD

STORM OF AUG. 29, 1975

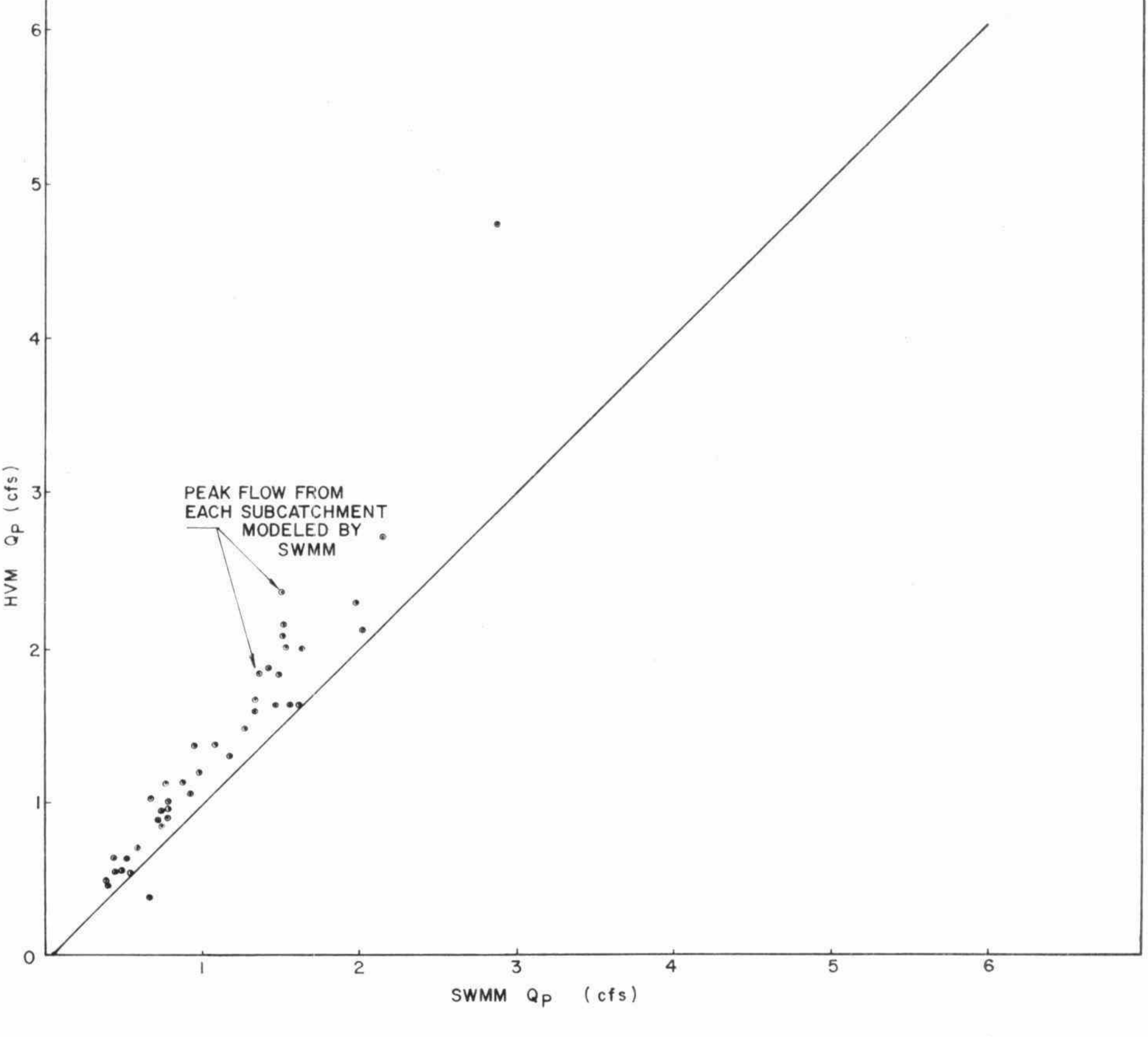
FIG. 4.24



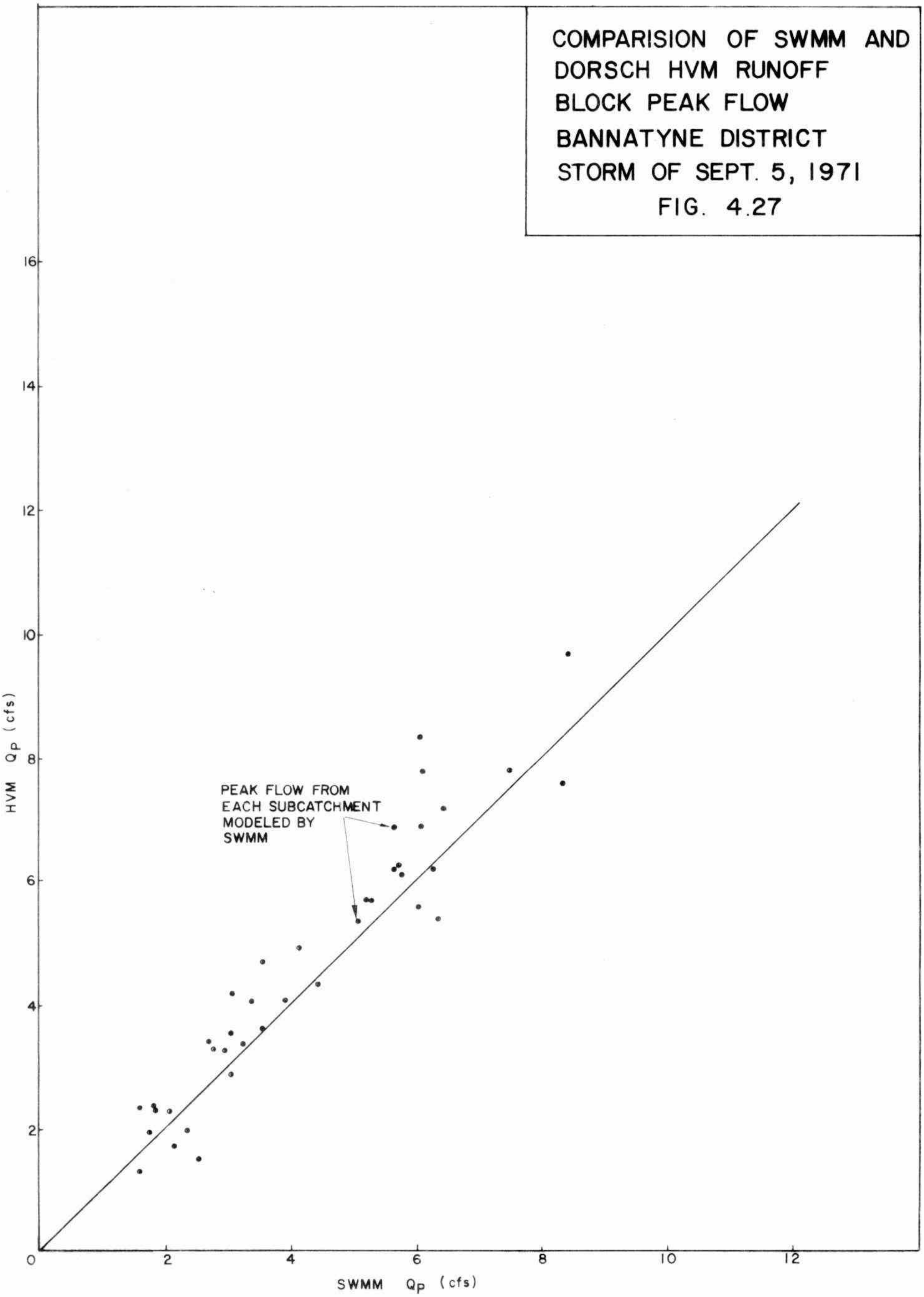
BRUCEWOOD

STORM OF NOV. 20, 1975

FIG. 4.25



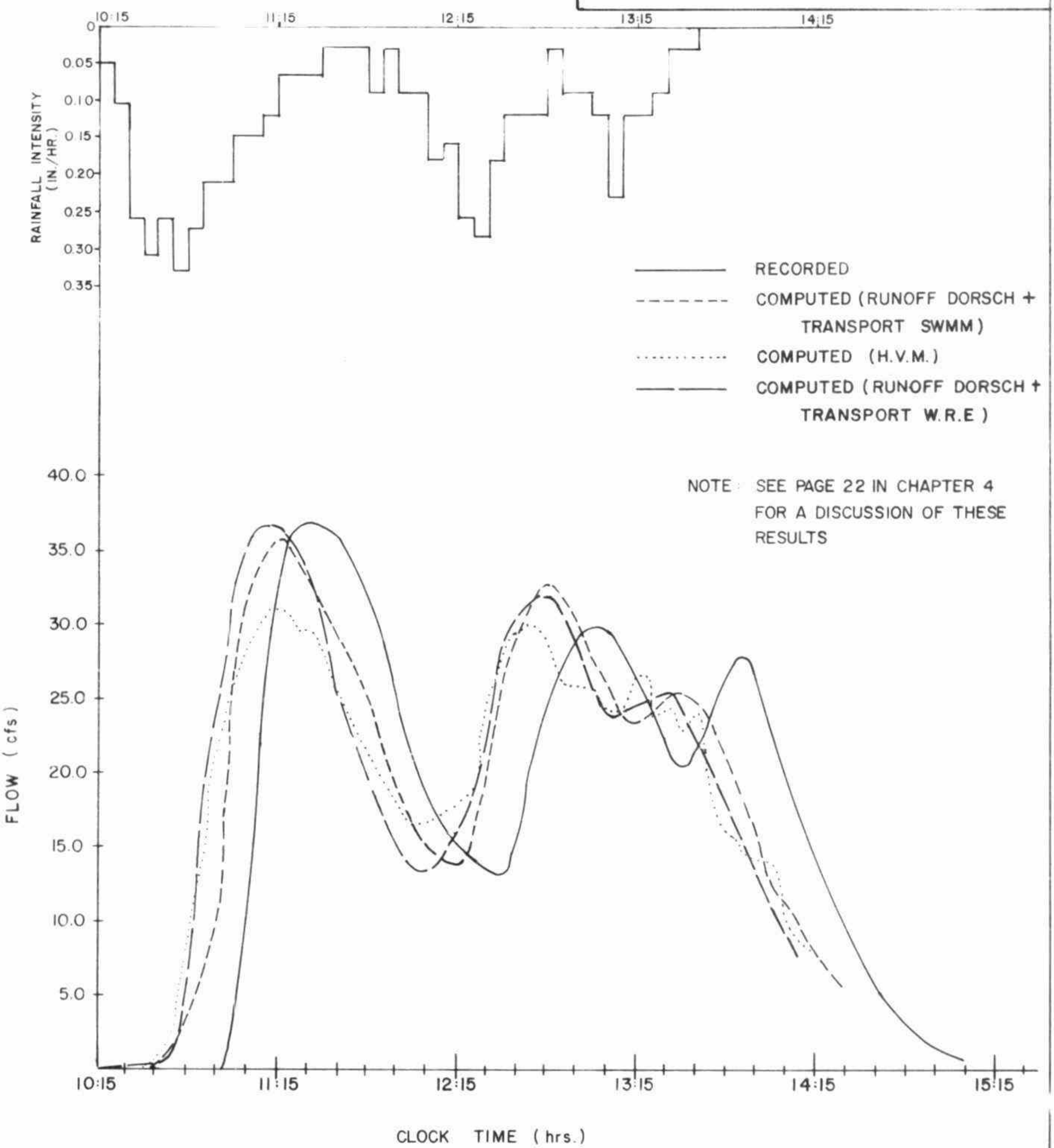
COMPARISION OF SWMM AND
DORSCH HVM RUNOFF
BLOCK PEAK FLOW
BANNATYNE DISTRICT
STORM OF JUNE 19, 1971
FIG. 4.26



COMPARISION OF SWMM AND
DORSCH HVM RUNOFF
BLOCK PEAK FLOW
BANNATYNE DISTRICT
STORM OF SEPT. 5, 1971
FIG. 4.27

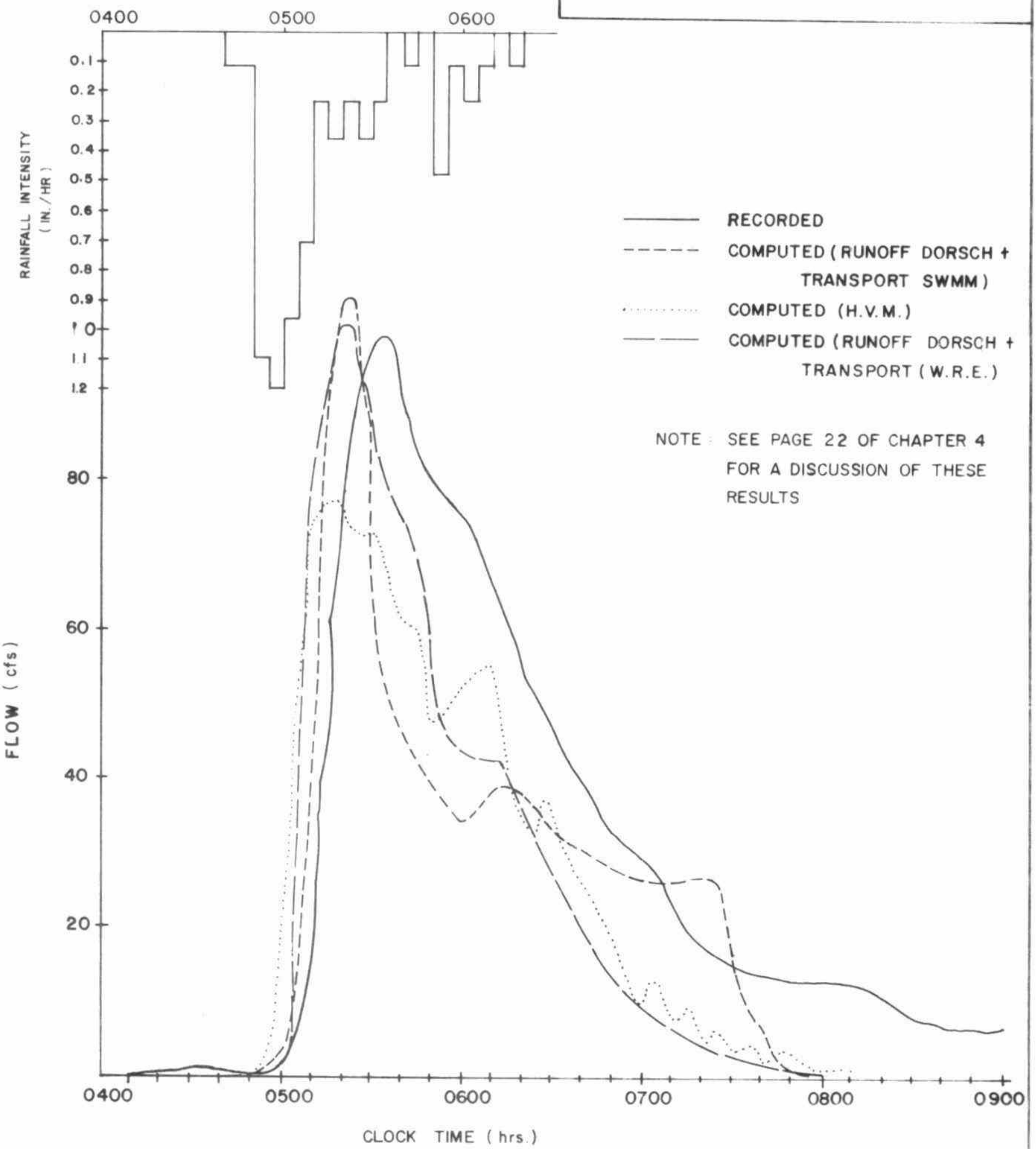
TRANSPORT SIMULATION
BANNATYNE DISTRICT
STORM OF JUNE 19, 1971

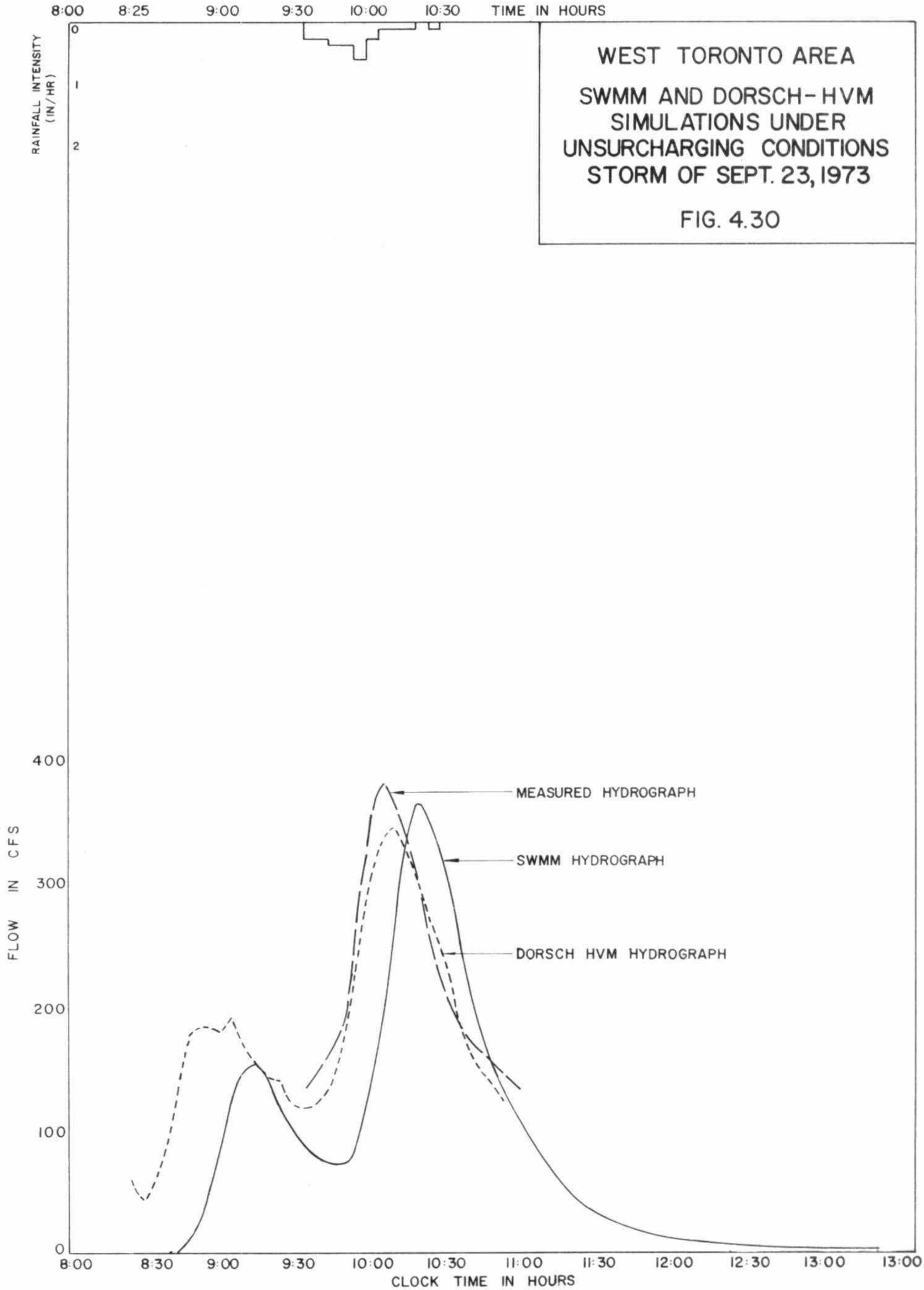
FIG. 4.28



TRANSPORT SIMULATION
BANNATYNE DISTRICT
STORM OF SEPT. 5, 1971

FIG. 4.29

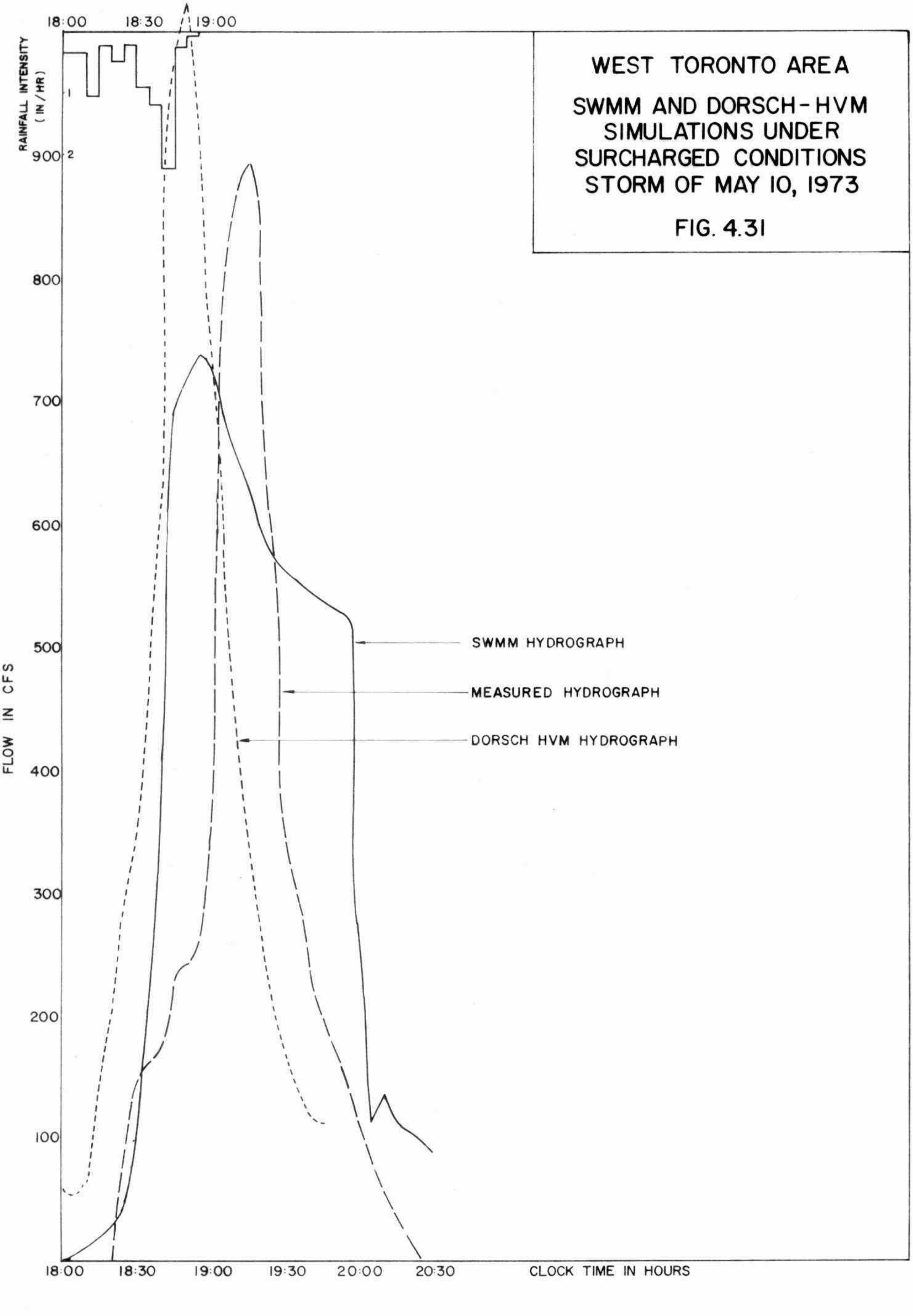




WEST TORONTO AREA

SWMM AND DORSCH-HVM SIMULATIONS UNDER UNSURCHARGING CONDITIONS STORM OF SEPT. 23, 1973

FIG. 4.30



WEST TORONTO AREA

SWMM AND DORSCH-HVM SIMULATIONS UNDER SURCHARGED CONDITIONS STORM OF MAY 10, 1973

FIG. 4.31

SWMM QUALITY SIMULATIONS FOR SELECTED CANADIAN WATERSHEDS

CHAPTER 5

CHAPTER 5

SWMM QUALITY SIMULATIONS FOR SELECTED CANADIAN WATERSHEDS

5.1 GENERAL

Studies described in the literature concerning urban stormwater quality have identified various factors contributing to stormwater pollution [1-11, 13, 16]. Generally pollutants can enter urban runoff from three major sources - the land surface (both pervious and impervious), catchbasins, and from sewers in a combined drainage system.

Water quality models for urban runoff can be classified into three main categories: empirical, statistical and analytical. A discussion and comparison of several existing urban runoff quality models is given in a literature survey, presented in Volume 2. A descriptive summary comparison of various models is presented in Table 5.1. In general, statistical and empirical models cannot be applied for watersheds, other than those for which they were developed, with a great amount of confidence in the results. Analytically based models may be applied in a wider variety of situations due to their more comprehensive nature. The SWMM quality routines are based principally on analytical formulations, but where the state of the art does not justify these, some empirical relationships have been built into the model. To date, testing of the SWMM quality models has been restricted to a few specific watersheds in the U.S. (for examples see [12] and Volume 2). In most cases their applications were limited to only few events and were directed towards preliminary calibration of model parameters and definition of default values.

The scope of this section of the study was to examine the applicability of the SWMM quality routines using available quality measurements from selected Canadian watersheds. The emphasis of the study has been placed on an assessment of model sensitivity to variations in quality parameters and default values, and on developing procedures for calibration. Some of the recommendat ions concerning modifications to the model subroutines and input data format

have been discussed with representatives of the University of Florida and have since been implemented, constituting options in the latest version of SWMM. The requirement for a more flexible approach to calibration resulted in the formulation of a generalized surface quality model, which is described in Chapter 10.

5.2 REVIEW OF SOME BASIC ASSUMPTIONS

The state of the art is such that there is not sufficient information available to model all the complex relationships between urban stormwater contaminants, their pollutional characteristics and the manner in which they are transported during storms [1, 2, 3]. Therefore, while the mathematical relationships used in the SWMM are the most comprehensive currently available, they are nevertheless based on simplifying analytical assumptions with some parameters determined by statistical means. The relationships presently used in the modelling of surface runoff in the SWMM are summarized in Figure 5-1. These basic equations, which are also used in other models, are discussed in more detail in the SWMM User's Manual [20]. However, a brief review is pertinent to the discussion contained in the following sections.

The initial mass of dust and dirt accumulated on the surface is estimated as a function of the number of dry days, and street cleaning frequency and efficiency prior to the storm event. The initial amounts of various pollutants, Po, in the dust and dirt are assumed to be in the proportions summarized in Table 5.2 for various land use classifications. The most important parameter used for surface runoff quality computations is the initial estimate of the total amount of each pollutant, Po, on the surface. High initial values of Po will give high initial washoff rates. Reference to the equations of Figure 5-1 indicates that variations in the number of dry days and street cleaning practices directly control the initial values of Po.

An exponential decay equation for surface washoff of pollutants assumes that the amount washed off in each time step is proportional to the total amount remaining on the surface. The rate of washoff is controlled by the runoff rate for each time step and an exponent, b. An average b value of 4.6 is assumed in the model (this is discussed in more detail in Section 5.3).

The most recent version of the SWMM, published by the University of Florida contains an optional method for computing suspended solids concentrations [20]. As indicated in Figure 5-1, the original method (ISS=0) is consistent with computations for other pollutants since it uses the exponential decay equation. However, an availability factor, computed as a function of runoff intensity, is also incorporated to account for the fact that at lower runoff intensities, insufficient energy is available to allow the solids remaining on the surface to be treated as completely available for washoff. For values of runoff intensity greater than 0.7 inches per hour, the availability factor is assumed to be 1.0 (i.e. all solids are available for washoff). The availability factor is used only for suspended solids computations.

The second optional method (ISS=1) for computing suspended solids is an empirical equation developed from San Francisco data during the initial SWMM development [12]. The physical significance of many parameters is difficult to assess. However, three major underlying assumptions are that:

(a) The amount of SS removed during each time step is essentially a function of the <u>ratio</u> of the amount remaining on the ground at the beginning of the time step (Po) divided by the initial amount on the ground at the beginning of the simulation period, (Poi).

(b) The amount removed is a function of a removing coefficient, CC, which varies from 0.9 to 0.25, decreasing over a period of about 5 hours from the start of the quality simulation.

(c) As programmed, the equation could be modified to operate over 3 ranges of runoff intensity. (The reader is referrred to Figure 5-1 for clarification). This assumption can sometimes result in some discontinuity in the computation of the SS pollutograph.

For DWF computations, either average values of SS, BOD and coliform concentrations can be supplied by the user or these can be estimated in the model, based on input data describing population and land use characteristics [12]. The program also allows for the hourly variation of these parameters to be modelled, either by using programme default values, or by supplying hourly rates. Deposition, scour and pollutant decay can also be modelled in the TRANSPORT block.

5.3 SENSITIVITY ANALYSIS

The SWMM stormwater quality simulations are dependent upon several parameters. Some of these must be supplied by the user while some may be supplied as an option and are also built into the model. The internal default values provided in the model are generally based on average measurements.

A sensitivity analysis involves the systematic independent variation of variables used in the computations in order to ascertain the effect of such variations on the results and to decide whether or not the model responds in a logical manner to these changes. The sensitivity analysis indicates how changes in specific variables will affect the results and so a knowledge of model sensitivity and a feeling for the allowable range over which the more influential parameters may be varied can lead to a logical adjustment of the model and to better simulations.

The relative sensitivity of the model to changes in the following parameters was assessed in this study in order to assess the potential for calibration:

(a) the exponent, b, in the exponential washoff equation (see Figure 5-1).

(b) the use of the two available options for SS computations - empirical (ISS=1) and exponential (ISS=0).

(c) street cleaning frequency and number of dry days, catchbasin B.O.D.

(d) pipe slopes in the TRANSPORT quality computations.

(e) the specific gravity of solid particles in TRANSPORT quality computations.

A hypothetical test area of 15 acres was used in the sensitivity analysis for the surface quality routines (Figure 5-3).

5.3.1 *Sensitivity to Washoff Exponent "b"*

The developers of the model defined the exponent, b, used in the washoff equation (see Figure 5-1) in the following manner:

"it was assumed that a uniform runoff of 1/2 inch per hour would wash away 90 percent of the pollutant in one hour". This leads to a value of b of 4.6. "Other assumptions for determing b could be made by this original assumption has proven satisfactory in all test applications to date for urban areas" ([12], p. 178).

This assumption was examined by plotting b versus runoff intensity for various removal rates as shown on Figure 5-2. Reference to this figure indicates that a value of b = 4.6 appears to be a "representative" value. For runoff intensities in excess of 0.7 inches/hour, b = 4.6 gives removal rates 100% in one hour. For smaller runoff intensities, and short duration storms, it is possible

that a significant amount of pollutant may remain on the ground at the end of the storm if a value of b of 4.6 (or less) is used. For a given rainfall intensity, higher values of b will result in a greater amount of total pollutant removal in the first few time steps and would hence give higher initial pollutant concentrations.

5.3.2 *Sensitivity to Type of Washoff Equation Used*

The model includes two equations for suspended solids calculations. These are referred to as (ISS=0) and (ISS=I) and may be written as:

$$POFF = AV * Po\ (1-e^{-b.r_1.\Delta t}) \text{ --------------(ISS=0)}$$

and

$$POFF = \frac{Po}{Po_i} * CC_t * (A*E + B*E^D) \text{ -------------(ISS=I)}$$

where POFF = pollutant load washed off in time step, Δt

AV = an availability factor (see Figure 5-1)

Po = surface pollutant load at start of time step

r_1 = runoff rate

b = constant (= 4.6 in SWMM)

Po_i = initial pollutant load

CC_t = a removing coefficient (maximum = 0.9)

A,E,B,D = empirical coefficients

An inspection of the equations suggests that for low or moderate values of Po (ISS=I) will tend to give a greater value for POFF in the first time step, unless the availability factor, AV, equals 1.0 since the initial ratio Po/Po_i will always be unity. (The value of AV used in SWMM is calculated from the runoff rate and is usually much smaller than 1.0). Also, as Po increases, due to higher pollutant loading rates or longer periods of pollutant accumulation, the differences between the two options might be expected to decrease. For large values of Po the exponential equation might be expected to give higher initial washoff rates since a ratio controls the washoff in the empirical equation.

The simulation results for the hypothetical area presented in Figure 5-4 indicate that for SS computations when the initial surface loading is relatively small, the empirical equation predicts concentrations which are significantly higher than those given by the exponential equation. However, for a larger initial surface loading, the exponential decay equation gives higher concentrations. The use of the exponential equation for SS computations is consistent with computations for other pollutants. However, as discussed in section 5.4 for low initial pollutant loadings (as a rule of thumb, say lower than 10-15 dry days prior to the storm), calibration to measurements may be possible only by using the empirical equation.

5.3.3 *Sensitivity to Initial Pollutant Accumulation*

The influence of the initial pollutant accumulation on the simulation of pollutants washed off is also indicated in Figure 5-4 for the hypothetical test catchment of Figure 5-3. In this example, the default pollutant loading rates and compositions were assumed. The number of dry days preceding the storm was varied in order to simulate different initial surface pollutant loads. A cleaning frequency greater than the number of dry days was assumed to avoid modelling of street cleaning. The results indicate that the total amount of pollutants washed off during this idealized storm is approximately a linear function of the preceding number of dry days.

Obviously street cleaning reduces the amount of surface pollutant available for washoff and will consequently reduce concentrations in the runoff. The simulation results shown on Figure 5-5 indicate the affect of variations in the street cleaning frequency.

5.3.4 *Sensitivity to Catchbasin BOD Concentration*

The sensitivity of the BOD calculation to the BOD concentration in the catchbasins was examined by varying the catchbasin BOD concentration for the 15 acre hypothetical area. A typical catchbasin density of 1 per acre and a volume of 30 $ft.^3$ were assumed. The BOD concentration was varied from 25 to 150 mg/l for an accumulation period of 3 dry days. As indicated below in Table 5.3, the peak BOD concentrations and total loads vary by a relatively small amount with the catchbasin concentration. (See also [12], Table 5.18). However, for relatively low intensity storms where the initial surface loading is small, some calibration may be possible by varying the catchbasin BOD concentration.

TABLE 5.3

EFFECT OF VARIATION OF CATCHBASIN BOD

Catchbasin Concentration BOD (mg/l)	Peak BOD Washed Off (mg/l)	Total Mass of BOD (lbs)
25	14.104	9.06
100	17.227	9.58
150	20.879	10.60

5.3.5 *Sensitivity to TRANSPORT Block Parameters*

The sensitivity analyses performed for the TRANSPORT Block quality routines used data from the Bannatyne combined sewer district in Winnipeg (542 acres).

The accumulation of a bed of solids in the sewer during periods of dry weather flow was found to vary considerably depending on the pipe slope and number of dry days. For example, pipe slopes in the order of .0001 ft./ft. resulted in simulated accumulations up to 100 times greater than those using pipes with slopes of the order of .001 ft./ft., depending on

pipe flow, diameter and length.

According to the procedure used in the model, the accumulation of solids in the sewers is directly related to the number of days during which no stormwater flow occurred. Thus, it is evident that the number of dry days is also an important parameter for combined systems, as illustrated in Table 5.5. Variations in the number of dry days modelled for combined systems can help to show the effects of sewer flushing as a pollution control method.

Routing and decay of non-conservative pollutants is modelled in the TRANSPORT conduits. However, "some doubt exists as to the significance of quality change, other than mixing, during rainfall-runoff periods. It has been suggested that the time of transit through the sewer system is too short to consider bacterial decay or reduction of BOD with time", ([12], p. 216). The effect of changes in the rate constant for decay on the percentage depletion of BOD has been shown to be relatively small ([12], Figure 13-2).

The sensitivity to the BOD decay rate was investigated by reducing the decay coefficient, D, from 0.20 to 0.0. The Bannatyne June 19 storm was run for these two values (simulation period 400 minutes), and the total amounts of BOD flushed out were 5,064 and 5,050 lbs. respectively, a difference of less than 0.5%. The difference between measured and computed BOD values is much greater than this, and hence D has essentially no effect.

The specific gravity of the solid particulate material in the dry weather flow in the TRANSPORT quality routine is fixed at 2.7. The effect of reducing this to 2.5 was investigated using the storm of June 19, 1971 for the Bannatyne district. A reduction in the total accumulated suspended solids load in the pipes from 6030 to 4538 lbs was observed. However, specific gravity is not considered to be useful for calibration purposes, unless the total solids discharge is consistently overestimated, since a value of 2.7 appears to be the maximum realistic value that could be used.

5.3.6 *Conclusions of Sensitivity Analysis*

In general, the SWMM quality model appears to respond in a logical manner to variations in input parameters. However, neither option for the computation of suspended solids from surface runoff seems generally applicable and the model is somewhat difficult to calibrate. This is due both to its complexity and the fact that some values that could logically be varied are fixed in the program.

5.4 SWMM SURFACE RUNOFF QUALITY SIMULATIONS

During the study period, there were very few Canadian data available for the study of surface runoff quality. However, for testing purposes, the preliminary data from the Brucewood catchment, located in North York, Toronto, were used. The catchment drains an area of 48 acres with an average imperviousness of about 47%. The predominant land use in the area is the modern single family unit. Figure 5-6 illustrates the detailed schematization and pipe network used to model the Brucewood catchment. The Brucewood quantity and quality monitoring programme and associated measurement problems are fully documented elsewhere [21]. Measured quality parameters include SS, BOD and coliforms. Various problems with the original automatic water quality sampler were encountered early in the measurement programme. Therefore, quality samples were sometimes taken manually prior to the summer of 1974. Consequently, for several events, quality samples are not available at the beginning of some of the storms. All available coliform measurements were collected manually.

For the purpose of this study, six storms were selected for comparison of measured data with quality simulations using SWMM. The number of quality measurements for other storms were generally insufficient for any worthwhile comparisons with simulated values. Of the storms selected, three had maximum rainfall intensities less than 0.60 inches per hour and only the storm of September 11, 1975 had a peak rainfall intensity greater than 1.0

inch per hour. As indicated in Table 5.4, measured peak flows ranged from 4 to 25 cfs. Also summarized in Table 5.4 are the total flow and pollutant amounts recorded during the quality sampling period.

Initially simulations were carried out for the six storms using the SWMM default values for the dust and dirt loading rates and pollutant composition. The procedure outlined in the User's Manual was followed to determine the antecedent number of dry days [20]; that is, prior days were counted as "dry" if the total volume of the antecedent rainfall was less than one inch.

The general trends observed in these initial runs are summarized below and form the basis for subsequent model adjustments.

(a) For events with a low number of antecedent dry days, suspended solids concentrations were severely underestimated when using the exponential equation (ISS=0). On the other hand, a more reasonable estimate was obtained for the storm of September 11, 1975, with 12 dry days.

(b) BOD concentrations were generally slightly underestimated when the exponential equation was used for suspended solids computations. (The model considers 5% of the computed SS to contribute to BOD). However, simulated BOD concentrations were much closer to measured BOD values than were the simulated SS to measured values.

(c) Simulated coliform concentrations were much higher than the measured values. (A programming error was detected in the coliform equations in the model. This has been rectified in cooperation with the University of Florida, May 1975).

These observations in conjunction with various calibration efforts and the results of the previous sensitivity analysis indicated that improved simulations could be achieved by making the following adjustments:

(a) Using the empirical (ISS=1) option for suspended solids calculations when initial surface pollutant loads are low.

(b) Supplying measured composition of pollutants wherever available. This included using the average measured value for catchbasin BOD concentration of 60 mg/l compared to the default value of 100 mg/l) for all the events and the dust and dirt accumulation rates measured prior to the events of August 11 and August 29, 1975. For these two events, the measured suspended solids fraction in the dust and dirt was taken as the percentage finer than the 30 mesh from the sieve analysis ([12], p. 185). The measured values for these storms are indicated on Figures 5-10 and 5-11. (The BOD composition measured in the dust and dirt was essentially the same as the default value).

(c) The MPN of coliforms in the dust and dirt was changed to 0.65 MPN per gram. This value was determined by trial and error simulations and used for the 3 events where coliform measurements were available for comparison.

The six storms were simulated with the input to the model adjusted as outlined. The final results only are presented on Figures 5-7 to 5-12. (Default values are used unless otherwise noted). For events with the antecedent number of dry days fewer than 10, the use of empirical equation (ISS=1) resulted in much better simulation of suspended solids concentrations than the exponential equation although estimated BOD concentrations were only marginally improved, if at all. For the storms of May 16, 1974, November 20, 1974 and September 11, 1975 (Figures 5-8, 5-9, 5-12) simulated and measured coliform MPN's are the same order of magnitude. These results

indicate that coliform estimates for separate sewer systems, using the corrected exponential decay equation, can give reasonable results compared to measured values provided some calibration is undertaken.

In general the results of these adjusted simulations indicate that pollutographs computed using the SWMM are of the same order of magnitude as the measured values.

5.5 SWMM COMBINED SEWER QUALITY SIMULATIONS

Comparisons of measured and simulated pollutographs have been carried out using data from the Bannatyne sewer district of Winnipeg. This district is served by combined sewers draining an area of about 542 acres. The predominant land uses are commercial and residential in about equal proportions. Only 5 storm events were considered suitable for simulation.

BOD and SS were the only parameters recorded in the Bannatyne area. Peak suspended solids and BOD concentrations were 154 to 9,294 mg/l and 25 to 1,720 mg/l respectively. These concentrations, which are relatively high, are attributed to scour of accumulated solids from the flat sloped pipe network.

Simulations were conducted using 41 subcatchments ranging in size from 5 to 23 acres. For all simulations, the catchbasin contribution of BOD was assumed to be 100 mg/l and the catchbasin volume was assumed to be 20 cubic feet [20]. As a result of discussions with the City of Winnipeg Works personnel, no street cleaning prior to each of the storms was assumed. Preliminary simulations indicated that the DWF generated by the model as a function of population and land use statistics significantly underestimated the measured DWF. Therefore, in order to reflect the measured data, an average DWF of 5.6 cfs with SS and BOD concentrations of 558 mg/l and 391 mg/l respectively were used (as determined from available measurements). The importance of accounting for the DWF in the simulation is indicated

in Figures 5-13 to 5-17 which compare: simulated pollutant concentrations from surface runoff alone and simulated concentrations accounting for both surface runoff and DWF, with measured pollutographs. The simulations for surface runoff alone shown on the figures correspond to the option (ISS=I) for the SS computations.

In this particular area the contribution of pollutants resulting from the flushing of sanitary deposits in the combined sewers outweighs the surface contribution. Consequently the choice of the SS surface washoff equation has little effect on the outflow pollutographs. In the curves shown in Figures 5-13 to 5-17 the empirical option was used as most of the antecedent dry periods were less than 10-15 days. Two different methods for estimating the number of antecedent dry days were employed. The first method summed the previous days on which zero rainfall had occurred. The second method determined the antecedent condition by assuming the "dry" period extended back to the date where the cumulative total antecedent rainfall exceeded one inch. The second method always gives the number of dry days greater than or equal to the first method. Only the pollutographs based on the second method are given on Figures 5-13 to 5-17. A comparison of the total mass of pollutants washed off from the start of the storm to the end of the sampling period with the corresponding simulated loads is presented in Table 5.5. In general, the results of the simulation of pollutographs based on the second method for determining the dry period are quite good. However, simulated BOD concentrations are consistently lower than the measured values. This might be due to the fact that there is likely to be an additional contribution of BOD resulting from the solids settled in the sewers. Inclusion in the model of a factor representing this additional BOD would require some analysis of deposits from a variety of areas.

It would obviously not be possible to obtain reasonable simulations without allowing, in the modelling process, for substantial deposition of solids in the combined sewers. Field observations noted by engineering personnel in the City of Winnipeg have confirmed the occurrence of significant accumulation of solids deposits during dry periods.

5.6 CONCLUSIONS

(a) The state of the art of stormwater quality modelling has not advanced as far as that of quantity modelling. However it is possible to obtain simulated pollutographs similar to measured pollutographs, for both separate stormwater and combined discharges.

(b) The sensitivity analysis indicates that logical changes in input parameters are logically reflected in the simulated results. Consequently, the model can be regarded as being a useful tool for comparison of stormwater management alternatives. However, caution should be exercised in using the model for estimates of the absolute value of stormwater pollutant loads.

(c) Neither of the options provided in the SWMM for suspended solids computations are adequate over a wide range of antecedent conditions. When the initial pollutant loading is very high, the exponential equation will simulate higher SS concentrations than the empirical equation. Conversely when the initial pollutant loading is fairly low, the empirical equation will simulate higher SS concentrations. In general, the empirical equation (ISS=I) may facilitate calibration when the dry period preceding the event is less than 10-15 days.

(d) The Quality Model in the SWMM is rather difficult to calibrate, partly because of its complexity and partly because the values of certain "variables" are fixed in the program. When the primary concern is surface runoff a simplified approach may be justified. The application of a Generalized SWMM Quality Model is discussed in Chapter 10.

(e) Initial conditions have been shown to control the quality computations. Consequently, one event quality models should be used in conjunction with continuous long term simulation models. These would be used to establish initial conditions for the more detailed model. (This is discussed more fully in Chapter 13).

TABLE 5.1

COMPARISON OF WATER QUALITY FEATURES OF VARIOUS MODELS

Model	Type of Model	Quality Constituents	Dry-weather Quality	Storm-runoff Quality	Quality Routing In Channels	Sedimentation and Scour in Channels	Quality Reactions In Channels
Stanley [11]	Empirical	BOD	No	Empirical relationship	No	No	No
Arnett et al [19]	Semi-empirical	BOD, suspended solids, phosphates, nitrogen	No	Linear regression equations	No	No	No
AVCO Corporation [9]	Statistical	BOD, COD total and suspended solids, nitrogen, phosphates total and fecal coliforms	No	Regression techniques, linear and non-linear relationships	No	No	No
Sartor & Boyd [3]	Statistical	Total solids	No	Regression techniques	No	No	No
Brandstetter [25]	Statistical	7 arbitrary conservative constituents	Hourly, daily and seasonal patterns	Linear regression equations	Pure advection, mixing between successive time steps	No	No

TABLE 5.1 (cont'd)

Model	Type of Model	Quality Constituents	Dry-weather Quality	Storm-runoff Quality	Quality Routing In Channels	Sedimentation and Scour in Channels	Quality Reactions in Channels
Corps of Engineers (STORM) [15]	Analytical	BOD, suspended and settleable solids, nitrogen and phosphates	No	Non-linear function of catchment characteristics, pollutant accumulation and runoff	No	No, but land surface erosion by universal soil loss equation	No
Hydrocomp [14]	Analytical	17 constituents including water temperature	Inflow concentration provided as input data or non-linear function of inflow hydrograph	Non-linear function of catchment characteristics, pollution accumulation and runoff	Pure advection, weighted mixing between successive time steps	No	Various reactions and interactions
EPA 1-Original Metcalf & Eddy Version [12]	Analytical	BOD, suspended solids, coliforms, nitrogen and phosphates	Hourly, daily and seasonal patterns provided as input data or computed from land use	Non-linear function of catchment characteristics, pollution accumulation and runoff. Different equation used for pollutant removal	Pure advection, mixing between successive time steps	Suspended solids, considering particle size distribution	First-order decay without interactions for BOD

TABLE 5.1 (cont'd)

Model	Type of Model	Quality Constituents	Dry-weather Quality	Storm-runoff Quality	Quality Routing In Channels	Sedimentation and Scour in Channels	Quality Reactions In Channels
2-Water Resources Engineers Version [26]	Analytical	COD, Settleable Solids and oil and grease in addition to above constituents	Same as EPA	Equation solved in a different way; no availability factors for suspended and settleable solids	Same as EPA	Same as EPA	Same as EPA
3-WRE Version modified by University of Florida [20]	Analytical	Same as EPA	Same as EPA	Two methods for suspended solids computation	Same as EPA	Same as EPA also includes land surface erosion by the universal soil loss equation	Same as EPA
Dorsch Model [24]	Analytical	BOD, Settleable solids other pollutants currently being added	Included but not documented	Empirical washoff equation	Continuity	Not presently included	No pollutant decay
UCURM [13]	Analytical	BOD, Suspended and settleable solids	No	Non-linear function of catchment characteristics, pollution accumulation and runoff	Pure advection mixing between successive time steps	Suspended solids, considering partial size distribution	No pollutant decay

TABLE 5.2

DEFAULT VALUES FOR DUST AND DIRT ACCUMULATION USED IN SWMM[a]

Land Use	Dust and Dirt Accumulation lbs/day/100ft. Curb		Coliforms[c]	Susp. Solids	mg Pollutant Per gm of Dust and Dirt					
					Sett. Solids	BOD	COD	N	PO_4	Grease
1. Single Family	0.7	(2.0)[e]	1.3×10^6	1000.0 (111.0)[d]	100.0	5.0	40.0	0.48	0.05	1.00
2. Multiple Family	2.3	(2.0)[e]	2.7×10^6	1000.0 (80.0)	100.0	3.6	40.0	0.61	0.5	1.00
3. Commercial	3.3	(2.9)[e]	1.7×10^6	1000.0 (170.0)	100.0	7.7	39.0	0.41	0.7	1.00
4. Industrial	4.6	(6.0)[e]	1.0×10^6	1000.0 (67.0)	100.0	3.0	40.0	0.43	.03	1.00
5. Undeveloped[b] or Park	1.5	(1.0)[e]	0.0	1000.0 (111.0)	100.0	5.0	20.0	0.05	.01	1.00

a - Most values are based on 1969 APWA Report [8], and included in [20].

b - Values for undeveloped and park lands are assumed.

c - Units for Coliforms are MPN/gram.

d - Values used in the WRE version of SWMM [26].

e - Data from [27].

TABLE 5.4

COMPARISON OF POLLUTANT LOADS FOR BRUCEWOOD SEPARATE STORM DRAINAGE SYSTEM*

Storm	Flow (cfs) Peak Recor'd	Flow (cfs) Comp'd	Sampling Period (min.)	Flow (cu.ft.) Recor'd	Flow (cu.ft.) Comp'd	BOD (lbs.) Recor'd	BOD (lbs.) Comp'd	SS (lbs.) Recor'd	SS (lbs.) Comp'd	Total Coliform (MPN) Recor'd	Total Coliform (MPN) Comp'd
May 14/74	11.4	9.2	30	8619	5068	2.63	7.89 (2.45)	49.27	117.09** (9.55)***	-	-
May 16/74	4.5	2.7	70	7401	3853	4.06	9.72 (6.83)	77.66	77.47 (3.93)	0.72×10^{10}	0.37×10^{10}
Nov 20/74	8.7	11.9	35	10362	13402	12.65	33.74 (10.85)	137.18	587.38 (97.47)	2.4×10^{10}	4.03×10^{10}
Aug 29/75	5.3	5.1	88	5653	6801	1.31	9.13 (5.4)	21.7	78.19 (4.09)	-	-
Sep 11/75	25.0	22.0	80	49422	53649	10.04	61.06 (45.86)	345.2	875.14 (605.8)	-	-

* All total washoff amounts have been computed for the sampling period only. Flow and pollutant amounts have not been included for the storm of Aug. 11/75, since the recorded runoff rate is extremely low during the sampling period.

** ISS = 1

*** ISS = 0

N.B. It is not possible to present a similar comparison for the event of Aug. 11/75 due to limited data.

TABLE 5.5

SIMULATION RESULTS FOR BANNATYNE WINNIPEG

STORM	Recorded Flow cu.ft.	Computed Flow cu.ft.	Ratio*	BOD Recorded lbs.	BOD Computed lbs.	Ratio*	SS Recorded lbs.	SS Computed lbs.	Ratio*
June 19/71									
ISS=1, 8 days 1)	261,300	241,716	0.93	3051	2190	0.72	16,585	11,378	0.69
ISS=0, 8 " 2)					2078	0.68		9,148	0.53
ISS=0,11 " 3)					2138	0.70		11,734	0.70
July 15/71									
ISS=1, 5 days 1)	111,600	101,131	0.91	1450	1124	0.78	5,382	4,613	0.86
ISS=0, 5 " 2)					1114	0.77		4,318	0.80
ISS=0,12 " 3)					1161	0.80		9,147	1.69
July 17/71									
ISS=1, 2 days 1)	171,960	143,184	0.83	2285	1085	0.48	9,423	3,823	0.41
ISS=0, 2 " 2)					1012	0.45		2,453	0.26
ISS=0,10 " 3)					1120	0.49		9,196	0.98
July 28/71									
ISS=1, 8 days 1)	227,700	159,136	0.7	2224	1485	0.67	24,988	9,306	0.37
ISS=0, 8 " 2)					1434	0.65		7,886	0.32
ISS=0,11 " 3)					1479	0.67		10,484	0.42
Sept. 5/71									
ISS=1,16 days 1)	447,240	373,879	0.84	27,141	1616	0.05	84,857	26,205	0.31
ISS=0,16 " 2)					1620	0.06		27,121	0.32
ISS=0,39 " 3)					3116	0.11		64,798	0.77

* Ratio of computed to measured values

REFERENCE - CHAPTER 5

1. "Pollutional Effects of Stormwater and Overflows from Combined Sewer Systems - A Preliminary Appraisal", U.S. Department of Health, Education and Welfare, U.S. Public Health Service, November 1964.

2. McPherson, M.B. "Hydrologic Effects of Urbanization in United States", ASCE Urban Water Resources Research Program, Technical Memorandum No. 17, June 1972.

3. Sartor, J.D., and Boyd, G.B., "Water Pollution Aspects of Street Surface Contaminants". Environmental Protection Technology Series EPA-R2-72-081, November 1972.

4. Weibel, S.R., et al, "Urban Land Runoff as a Factor in Stream Pollution", JWPCF, 36, 914 (1964).

5. Bryan, E.H., "Quality of Stormwater Drainage from Urban Land", Water Resources Bulletin, 8, 578 (1972).

6. Whipple, W., et al, "Unrecorded Pollution from Urban Runoff", JWPCF, 46, 5, May 1974.

7. Pravoshinsky, N.A., and Gatillo, P.D., "Determination of the Pollutional Effect of Surface Runoff", International Association of Water Pollution Research, Prague, Czech., Advances in Water Pollution Research, 1969, PP. 187-195.

8. "Water Pollution Aspects of Urban Runoff", U.S. Department of the Interior, Federal Water Pollution Control Administration, WP-20-15, January 1969.

9. "Storm Water Pollution from Urban Land Activity", AVCO Economic Systems Corporation, FWQA Contract No. 14-12 187, FWQA Pub, No. 11034 FKL, April 1970.

10. Waller, D.H., "Pollution Attributable to Surface Runoff", Final Report to Central Mortgage and Housing Corporation, Ottawa, April 1971.

11. Stanley, P.H., "How to Analyze Combined Sewage Stormwater", Water and Wastes Engineering, April 1966.

12. "Stormwater Management Model", Metcalf and Eddy, Inc., and University of Florida, and Water Resources Engineers, Inc., Report to the EPA, I-IV 11024 DOC07/71 (1971).

13. "Urban Runoff Characteristics", Division of Water Resources Department of Civil Engineering, University of Cincinnati, Report to the EPA, 11024 DQU10/70 (1970).

14. Hydrocomp International, Inc., Hydrocomp Simulation Programming --- Operational Manual. Palo Alto, Calif. February, 1962.

15. Urban Runoff: Storage, Treatment and Overflow Model-STORM, Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, California, September 1973.

16. Loehr, R.C., "Characteristics and Comparative Magnitude of Non-Point Sources", JWPCF, Vol. 46, No. 8, August 1974.

17. Sawyer, C.N.: "Fertilization of Lakes by Agricultural and Urban Drainage", Jour. New Eng. Water Works Assn., 61, 109 (1947)

18. Sylvester, R.O.: "Nutrient Content of Drainage Water from Forested, Urban and Agricultural Areas". U.S. Dept. of Health, Education & Welfare, Publ. N. SEC-TR-W61-3, Cincinnati, Ohio, (1961).

19. Arnett, R.C., et al, "Diurnal Flow and Quality Patterns in a Combined Sewerage System", Paper Presented at the 46th Annual Water Pollution Control Federation Conference, Cleveland Convention Center, Cleveland, Ohio, September 30 to October 5, 1973.

20. Huber, W.C. et al, "Storm Water Management Model User's Manual - Version II", Report to the EPA March 1975.

21. Unpublished Draft "Report on the Brucewood Monitoring Programme, February 15, 1974 - December 31, 1974," James F. MacLaren Ltd., Prepared for Environment Canada/Ontario M.O.E., September 1975.

22. Shubinski, R.P., "Structure of the New TRANSPORT block EPA-SWMM", Unpublished paper presented at the SWMM User's Group Meeting, Gainesville, Florida, February 1975.

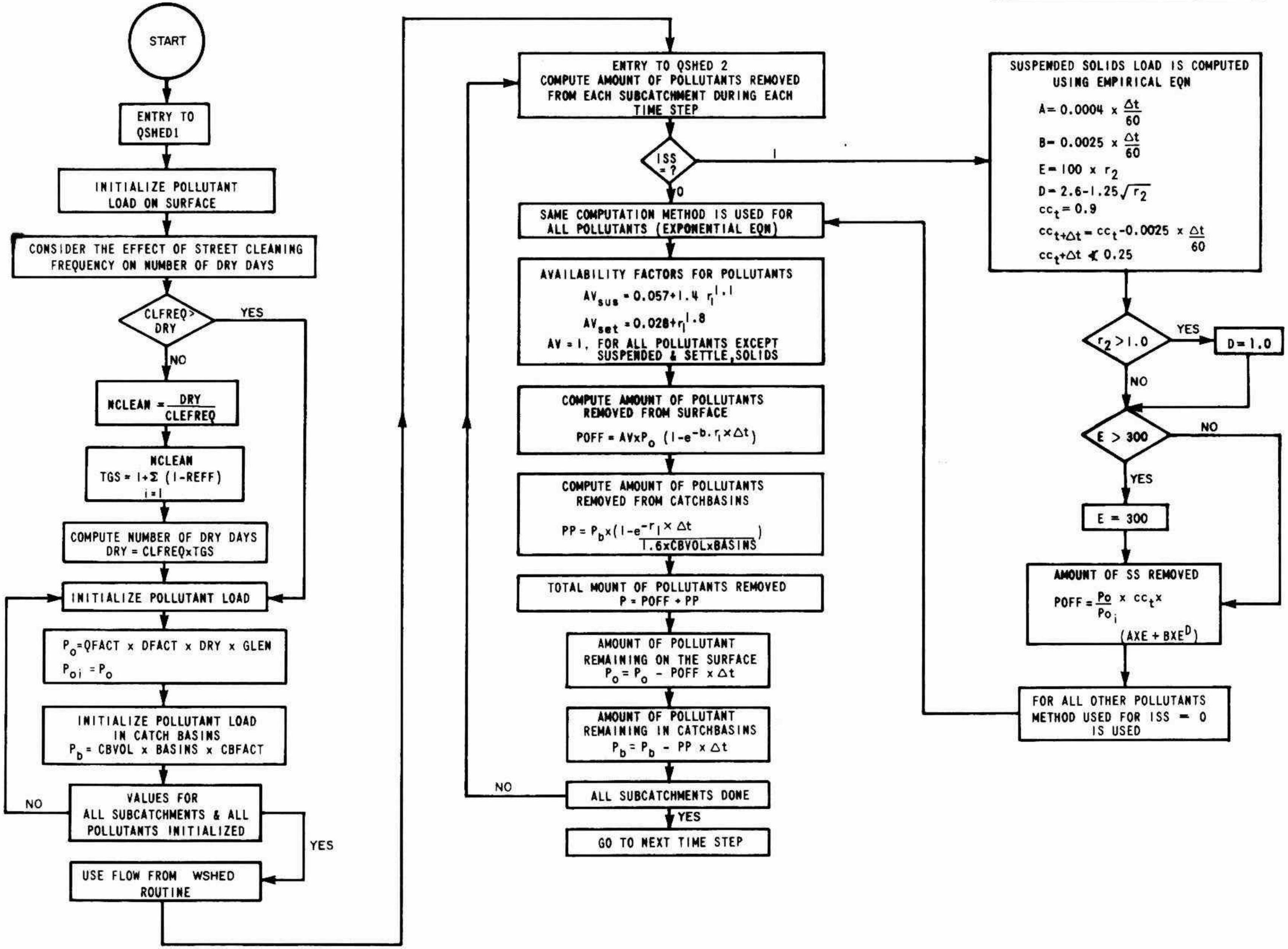
23. Shubinski, R.P. and Roesner, L.A. "Linked Process Routing Models", Proc. Symposium on models for Urban Hydrology, American Geophysical Union, April 1973.

24. Klym, H., Koniger, W, Mervius, F, Vogel, G., "Urban Hydrologic Process - Computer Simulation", Dorsch Consult, Munich Germany, and Toronto, Ontario.

25. Brandstetter, A., R., L., Engel and Cearlock, D.B., "A Mathematical model for optimum Design and Control of Metropolitan Wastewater Management Systems" Water Resources Bulletin 9, 6, December 1973.

26. Huber, W.C., "Memorandum - Differences Between Old and New Runoff Quality Models" May 28, 1974, Dept. of Env. Eng., University of Florida, Gainesville, Florida.

27. Huber, W.C., "Personal Communication" - University of Florida, 1975.



ALGORITHM OF THE SWMM SURFACE RUNOFF QUALITY MODEL

FIG. 5.1

QSHED1 = subroutine to initialize pollutant loads on the surface

P_o = total pollutant load on the ground surface at the beginning of time step

PO_i = initial pollutant load on the ground surface at the start of storm

QFACT = MG of pollutant per gram of dust & dirt

DFACT = dust & dirt loading rate for each land use (lbs/day/100 ft curb)

DRY = number of dry days

GLEN = curb length

P_b = total pollutant load in the catchbasins at the start of storm

CBVOL = catchbasin volume

BASINS = number of catchbasins in subcatchment

CBFACT = concentration of pollutants in catchbasins

QSHED2 = subroutine to comput the amount of pollutant removed.

ISS = method for calculating suspended solids

AV_{sus} = availability factor for suspended solids

AV_{set} = availability factor for settleable solids

AV = availability factor for pollutants other than SS and Set. Solids

r_1 = runoff rate in in/sec

r_2 = runoff rate in in/hr

cc = removing coefficient

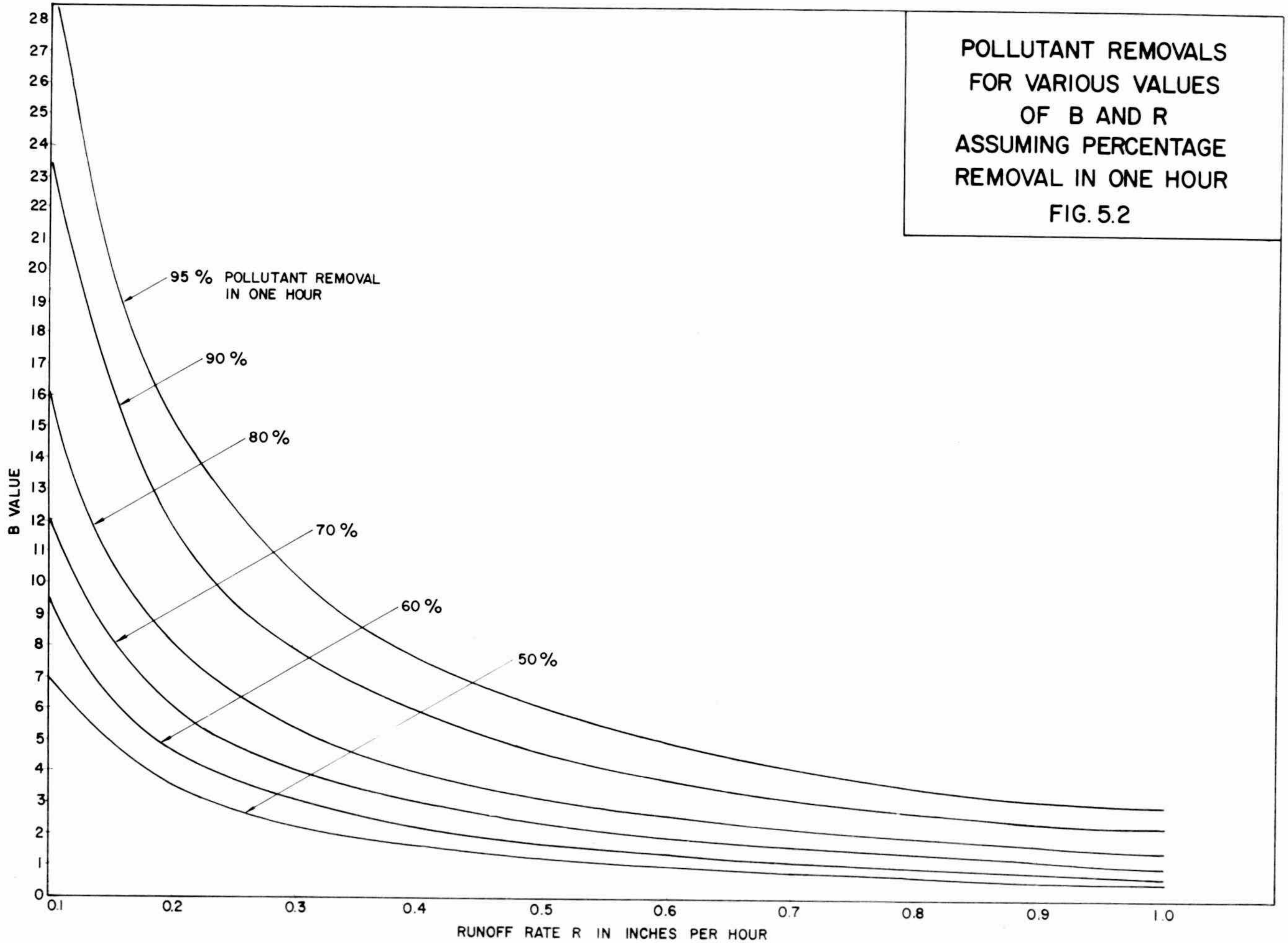
POFF = amount of pollutant removed from the surface during each time step

PP = amount of pollutant removed from catchbasin during each time step

CLFREQ = street cleaning frequency

REFF = cleaning efficiency of the equipment

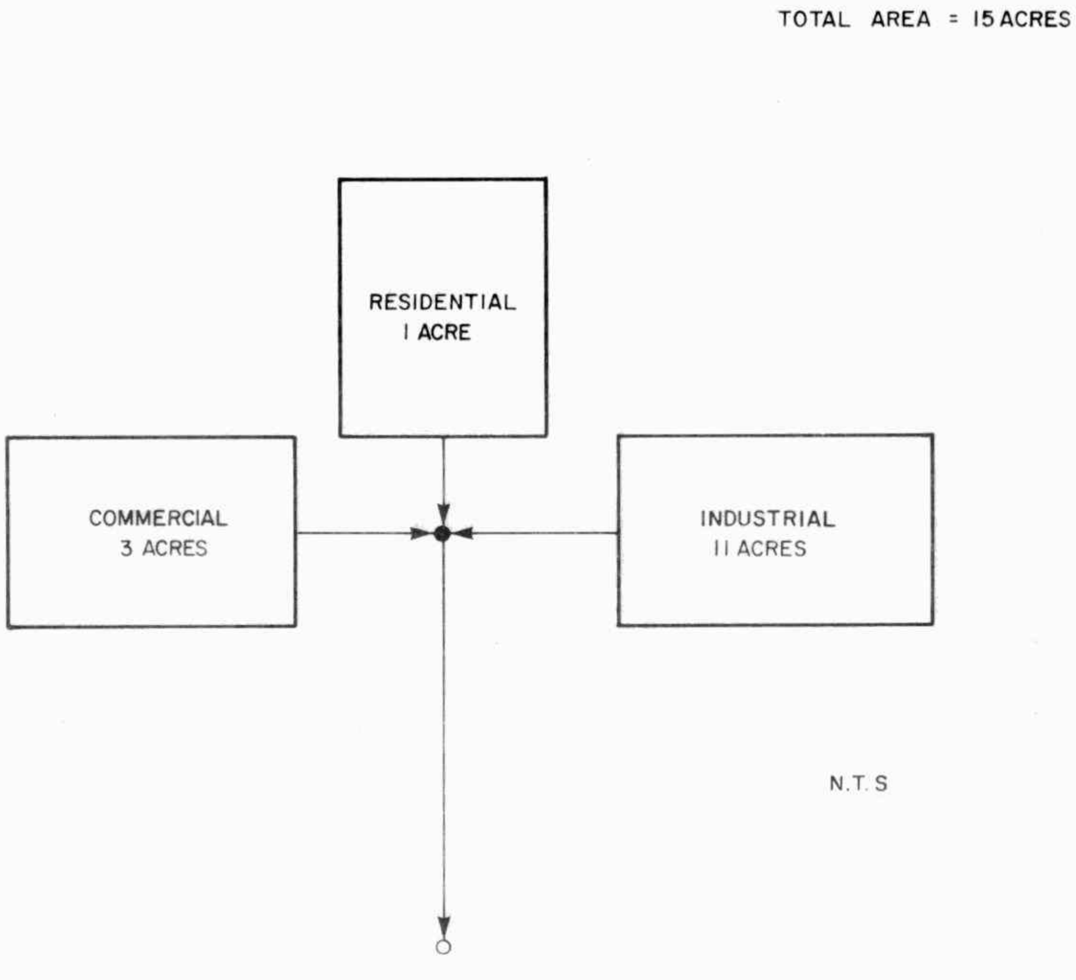
b = exponent coefficient

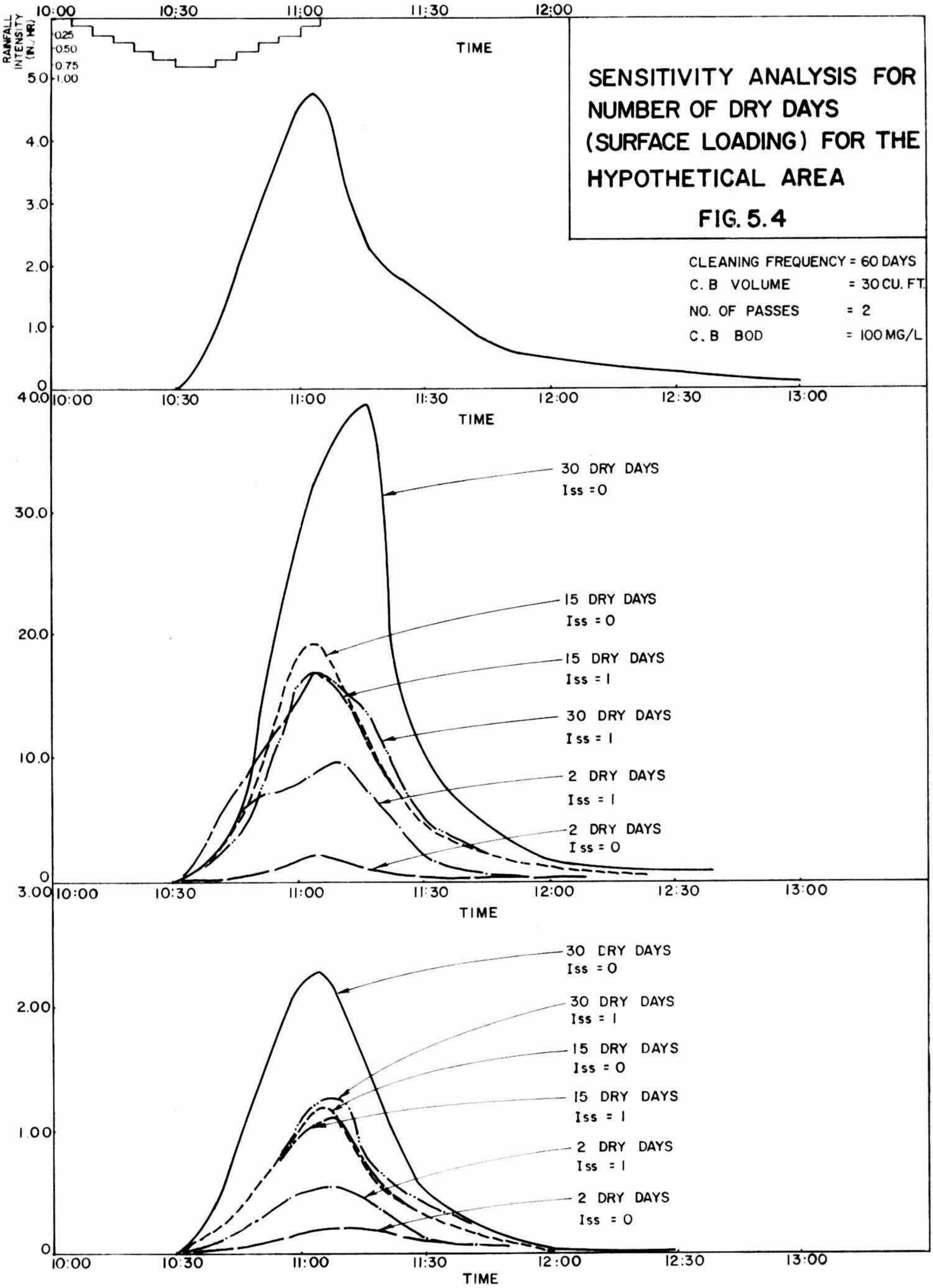


POLLUTANT REMOVALS
FOR VARIOUS VALUES
OF B AND R
ASSUMING PERCENTAGE
REMOVAL IN ONE HOUR
FIG. 5.2
95 % POLLUTANT REMOVAL
IN ONE HOUR
90 %
80 %
70 %
60 %
50 %
B VALUE
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0
RUNOFF RATE R IN INCHES PER HOUR

CONFIGURATION OF HYPOTHETICAL AREA USED FOR TESTING SENSITIVITY OF SURFACE LOADING

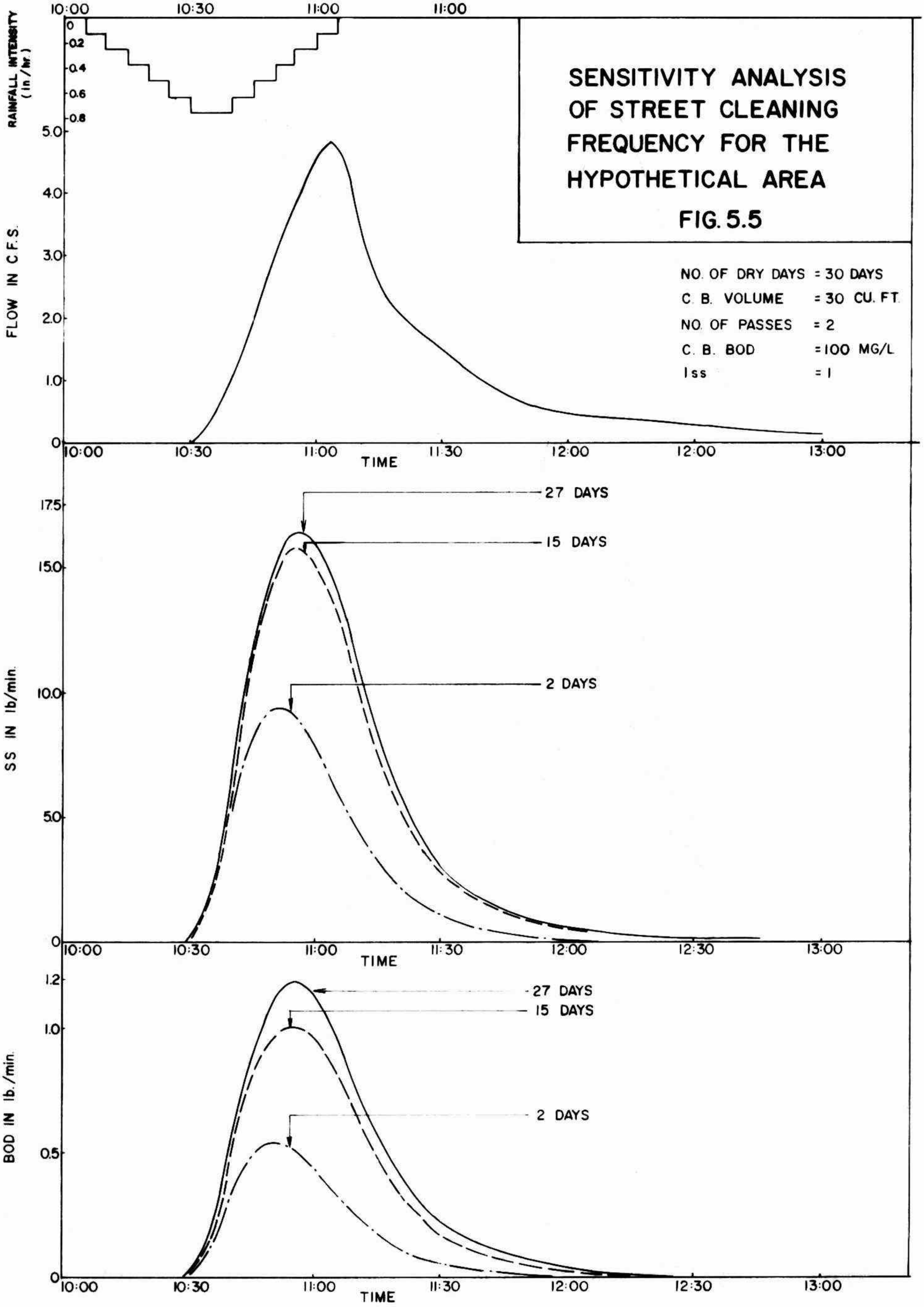
FIG. 5.3



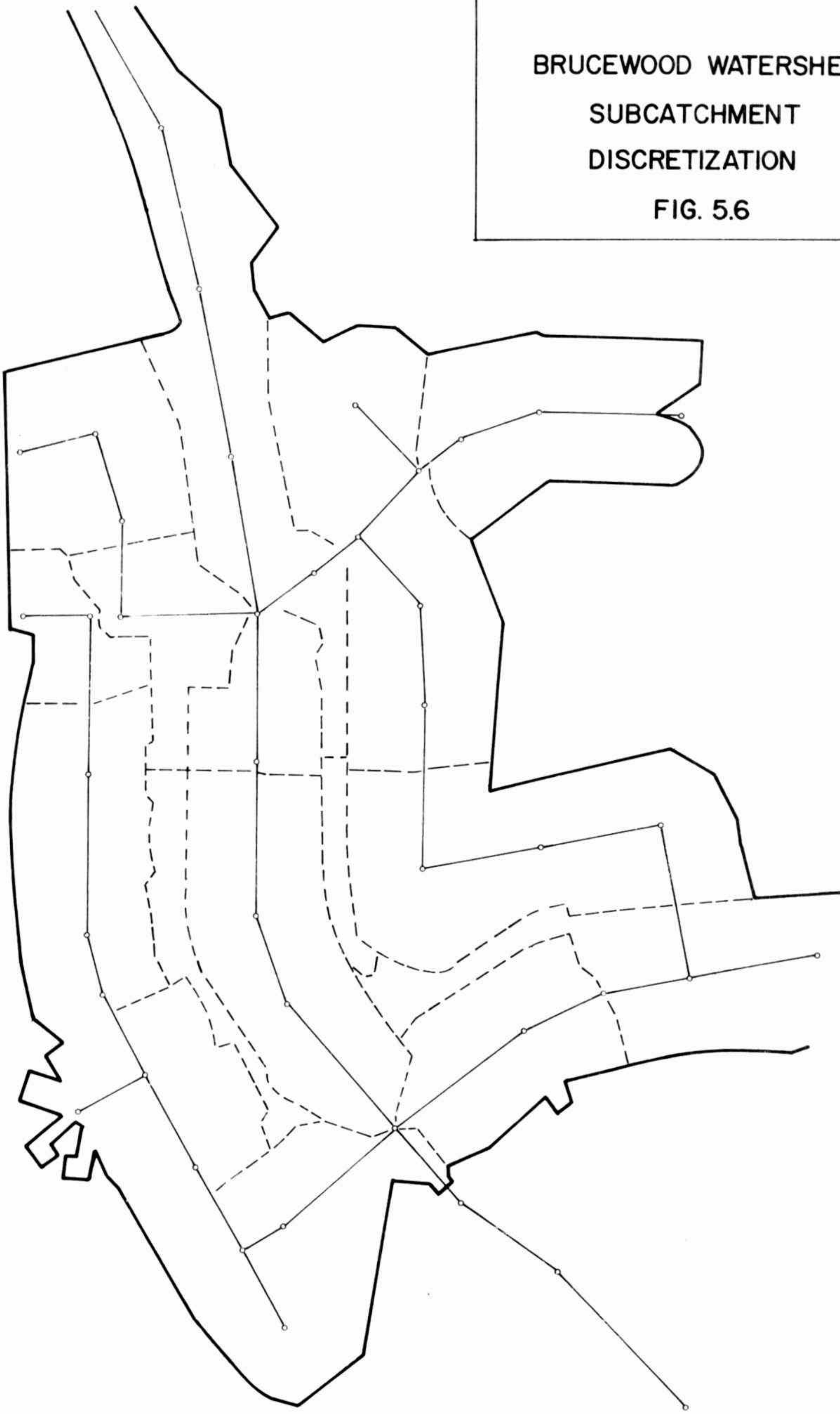


SENSITIVITY ANALYSIS FOR NUMBER OF DRY DAYS (SURFACE LOADING) FOR THE HYPOTHETICAL AREA

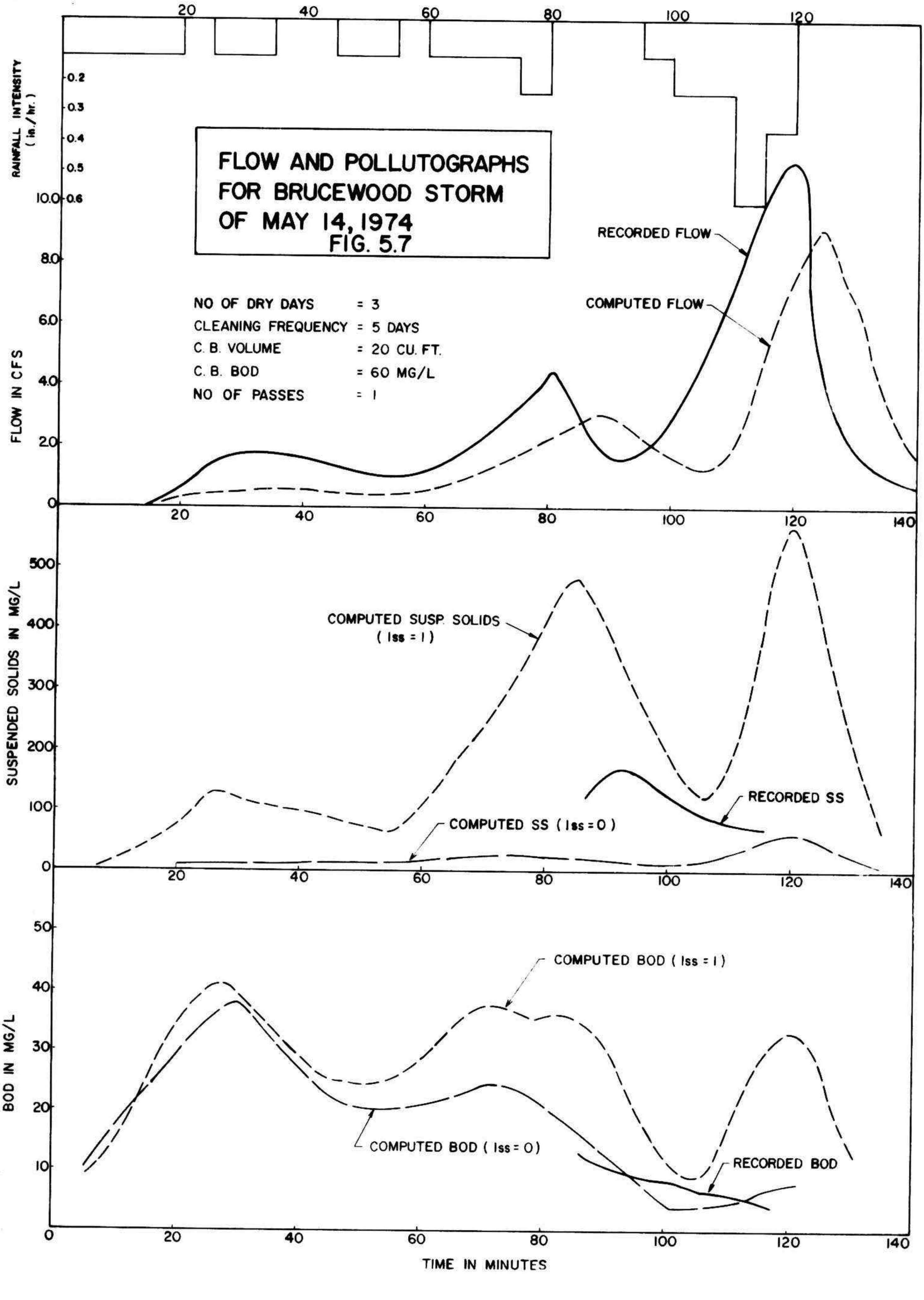
FIG. 5.4



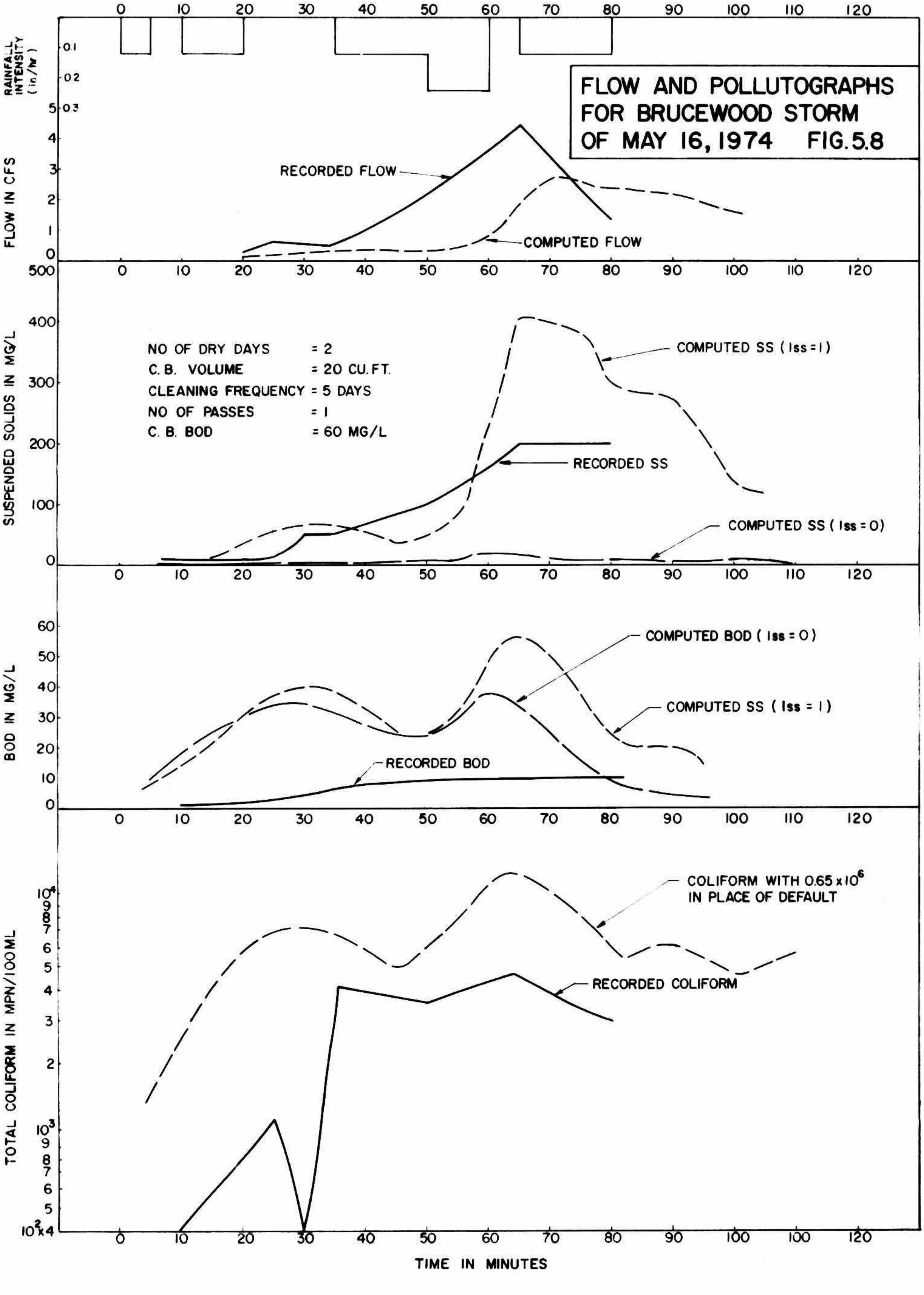
SENSITIVITY ANALYSIS OF STREET CLEANING FREQUENCY FOR THE HYPOTHETICAL AREA
FIG. 5.5
RAINFALL INTENSITY (in/hr.)
0
0.2
0.4
0.6
0.8
10:00
10:30
11:00
11:00
FLOW IN C.F.S.
5.0
4.0
3.0
2.0
1.0
0
NO. OF DRY DAYS = 30 DAYS
C. B. VOLUME = 30 CU. FT.
NO. OF PASSES = 2
C. B. BOD = 100 MG/L
Iss = 1
10:00
10:30
11:00
11:30
12:00
12:00
13:00
TIME
SS IN lb/min.
17.5
15.0
10.0
5.0
0
27 DAYS
15 DAYS
2 DAYS
10:00
10:30
11:00
11:30
12:00
12:30
13:00
TIME
BOD IN lb./min.
1.2
1.0
0.5
0
27 DAYS
15 DAYS
2 DAYS
10:00
10:30
11:00
11:30
12:00
12:30
13:00
TIME



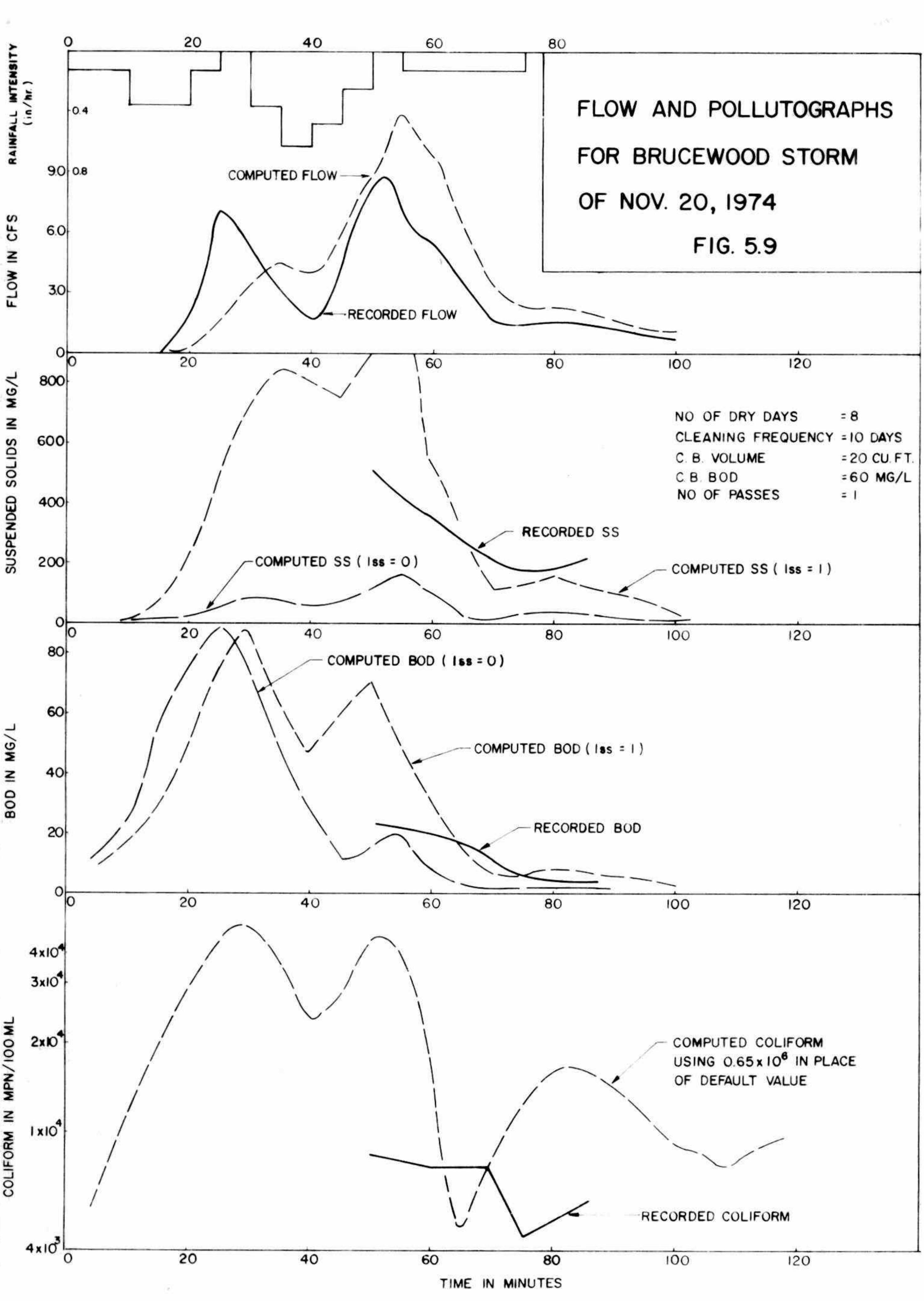
BRUCEWOOD WATERSHED
SUBCATCHMENT
DISCRETIZATION
FIG. 5.6



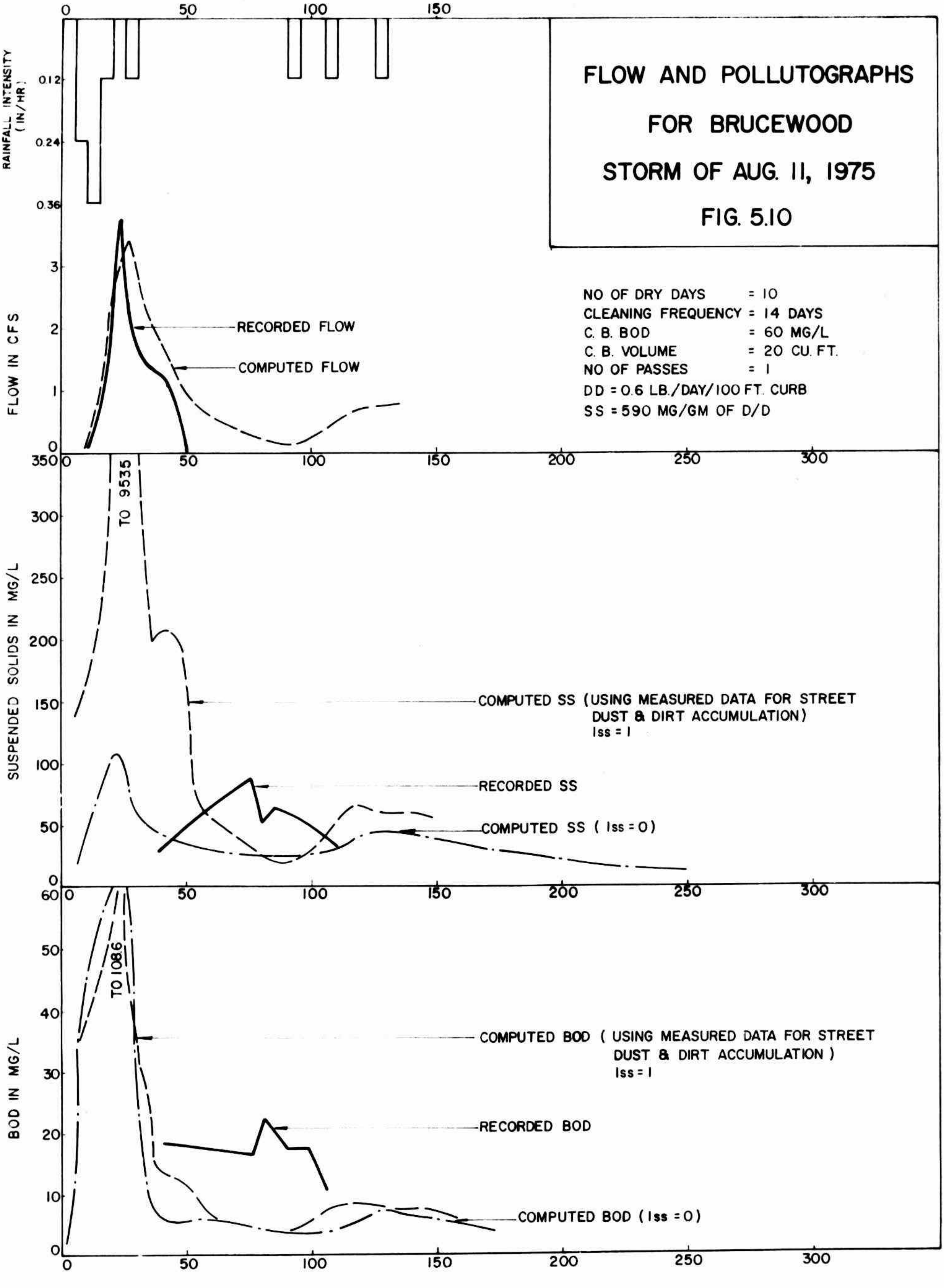
FLOW AND POLLUTOGRAPHS
FOR BRUCEWOOD STORM
OF MAY 14, 1974
FIG. 5.7
RAINFALL INTENSITY (in./hr.)
0.2
0.3
0.4
0.5
0.6
20
40
60
80
100
120
NO OF DRY DAYS = 3
CLEANING FREQUENCY = 5 DAYS
C. B. VOLUME = 20 CU. FT.
C. B. BOD = 60 MG/L
NO OF PASSES = 1
RECORDED FLOW
COMPUTED FLOW
FLOW IN CFS
10.0
8.0
6.0
4.0
2.0
0
140
SUSPENDED SOLIDS IN MG/L
500
400
300
200
100
COMPUTED SUSP. SOLIDS
(Iss = 1)
RECORDED SS
COMPUTED SS (Iss = 0)
BOD IN MG/L
50
40
30
20
10
COMPUTED BOD (Iss = 1)
COMPUTED BOD (Iss = 0)
RECORDED BOD
TIME IN MINUTES

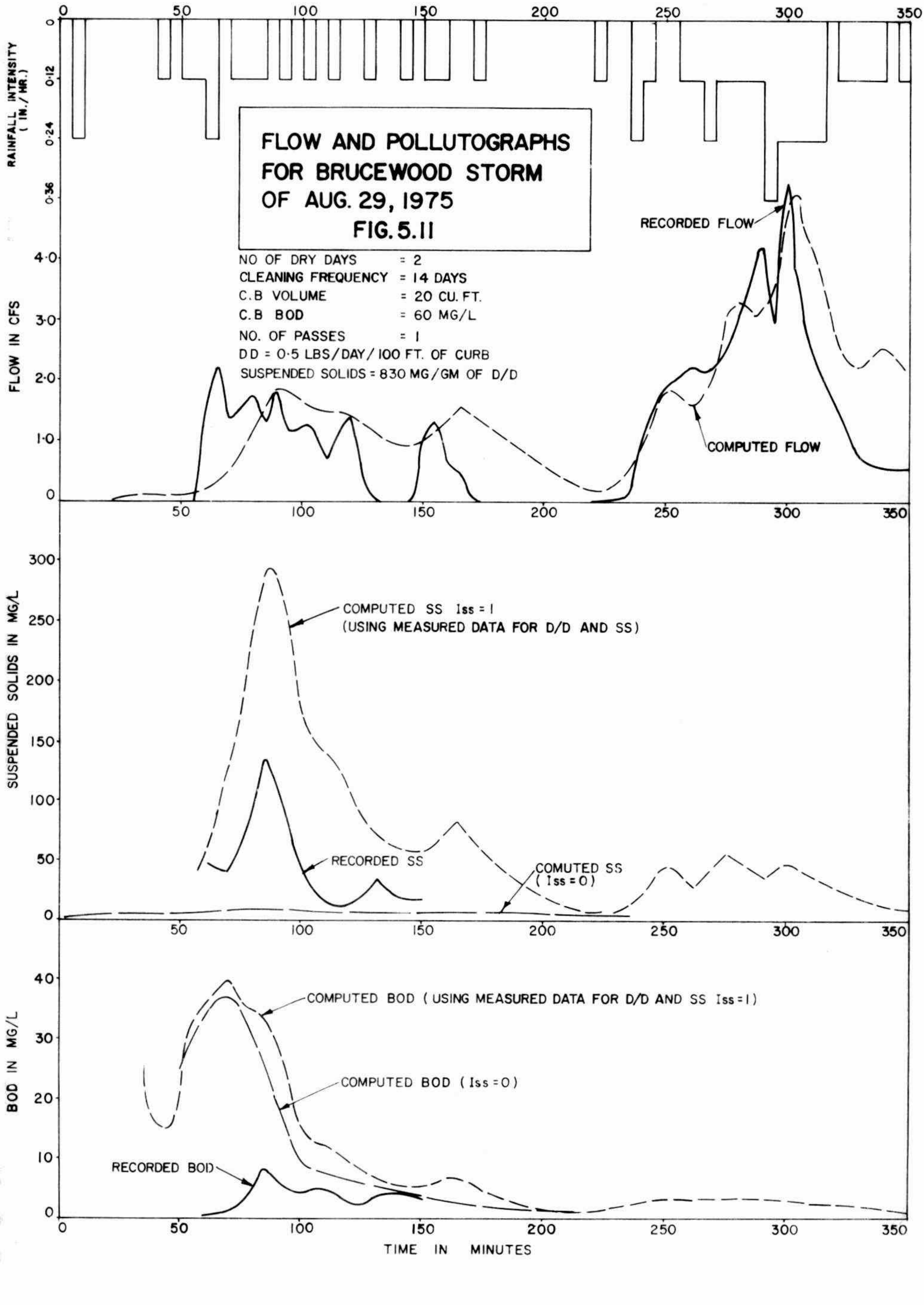


FLOW AND POLLUTOGRAPHS FOR BRUCEWOOD STORM OF MAY 16, 1974 FIG. 5.8
RAINFALL INTENSITY (in/hr)
FLOW IN CFS
RECORDED FLOW
COMPUTED FLOW
SUSPENDED SOLIDS IN MG/L
NO OF DRY DAYS = 2
C. B. VOLUME = 20 CU. FT.
CLEANING FREQUENCY = 5 DAYS
NO OF PASSES = 1
C. B. BOD = 60 MG/L
COMPUTED SS (Iss = 1)
RECORDED SS
COMPUTED SS (Iss = 0)
BOD IN MG/L
COMPUTED BOD (Iss = 0)
COMPUTED SS (Iss = 1)
RECORDED BOD
TOTAL COLIFORM IN MPN/100 ML
COLIFORM WITH 0.65 x 10^6 IN PLACE OF DEFAULT
RECORDED COLIFORM
TIME IN MINUTES

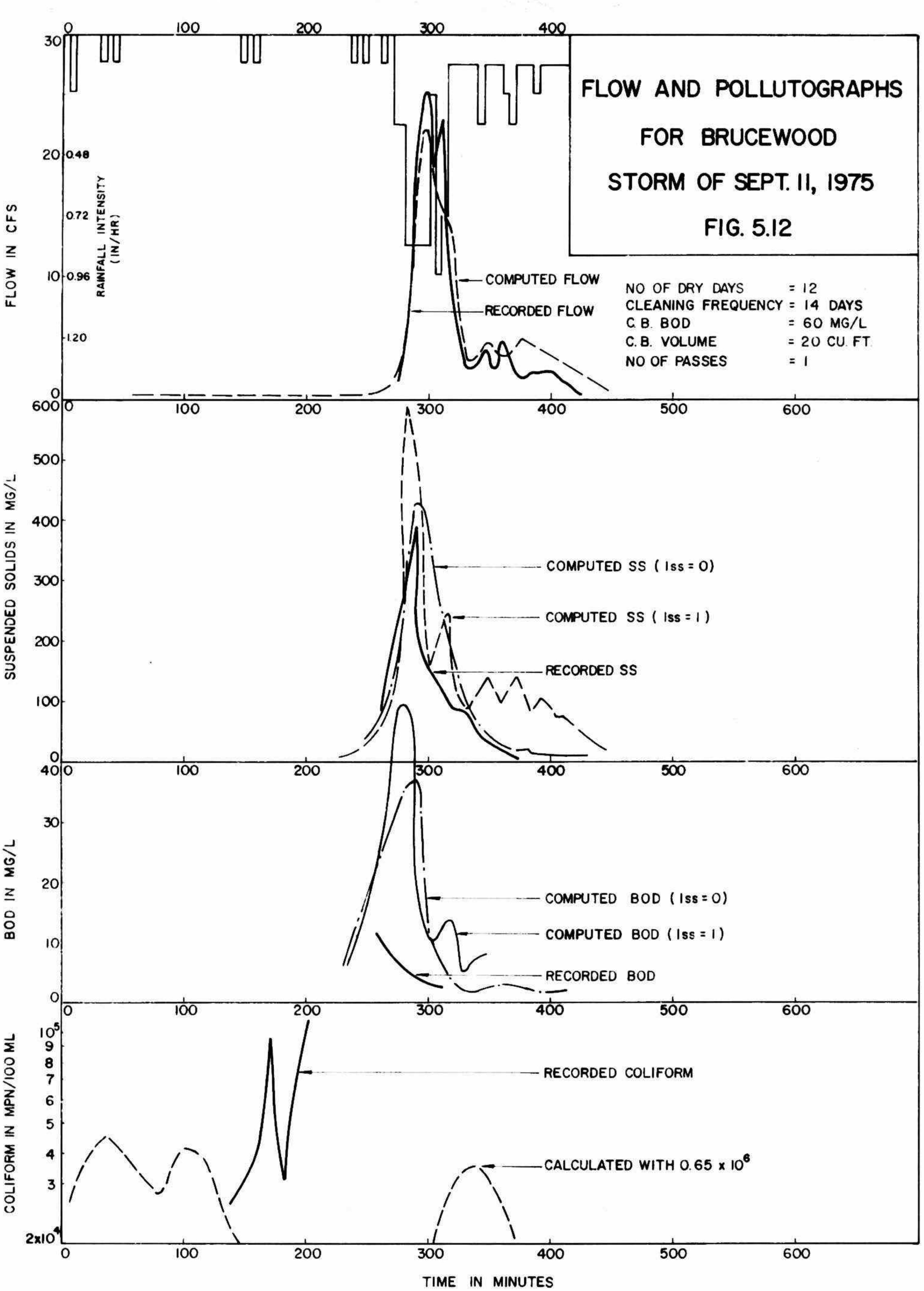


FLOW AND POLLUTOGRAPHS FOR BRUCEWOOD STORM OF NOV. 20, 1974
FIG. 5.9
RAINFALL INTENSITY (in/hr.)
FLOW IN CFS
COMPUTED FLOW
RECORDED FLOW
SUSPENDED SOLIDS IN MG/L
NO OF DRY DAYS = 8
CLEANING FREQUENCY = 10 DAYS
C. B. VOLUME = 20 CU. FT.
C. B. BOD = 60 MG/L
NO OF PASSES = 1
RECORDED SS
COMPUTED SS (Iss = 0)
COMPUTED SS (Iss = 1)
BOD IN MG/L
COMPUTED BOD (Iss = 0)
COMPUTED BOD (Iss = 1)
RECORDED BOD
COLIFORM IN MPN/100 ML
COMPUTED COLIFORM USING 0.65 x 10^6 IN PLACE OF DEFAULT VALUE
RECORDED COLIFORM
TIME IN MINUTES

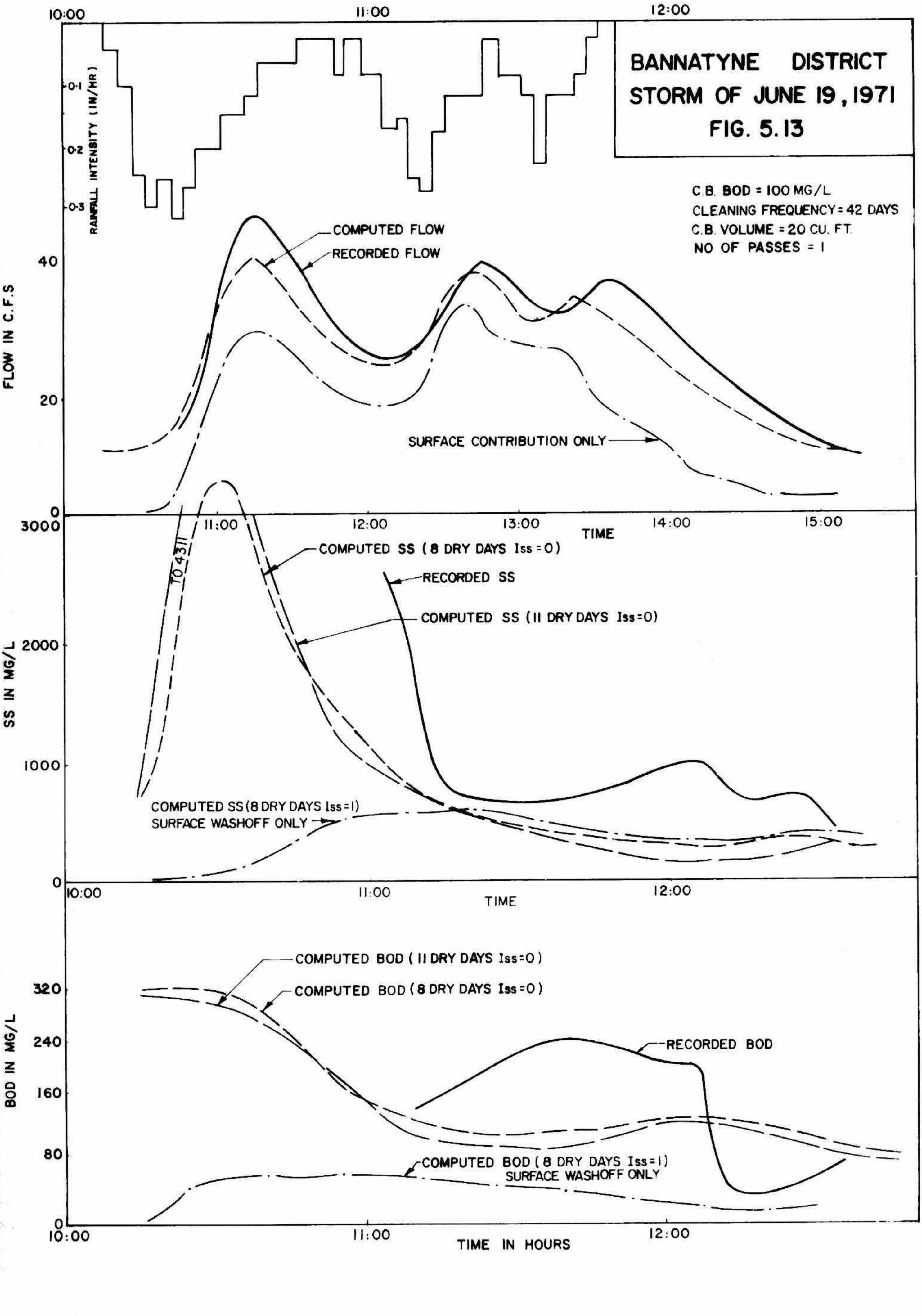

FLOW AND POLLUTOGRAPHS
FOR BRUCEWOOD
STORM OF AUG. 11, 1975
FIG. 5.10
NO OF DRY DAYS = 10
CLEANING FREQUENCY = 14 DAYS
C. B. BOD = 60 MG/L
C. B. VOLUME = 20 CU. FT.
NO OF PASSES = 1
DD = 0.6 LB./DAY/100 FT. CURB
SS = 590 MG/GM OF D/D
RAINFALL INTENSITY (IN/HR)
FLOW IN CFS
RECORDED FLOW
COMPUTED FLOW
SUSPENDED SOLIDS IN MG/L
TO 9535
COMPUTED SS (USING MEASURED DATA FOR STREET DUST & DIRT ACCUMULATION) Iss = 1
RECORDED SS
COMPUTED SS (Iss = 0)
BOD IN MG/L
TO 1086
COMPUTED BOD (USING MEASURED DATA FOR STREET DUST & DIRT ACCUMULATION) Iss = 1
RECORDED BOD
COMPUTED BOD (Iss = 0)
TIME IN MINUTES



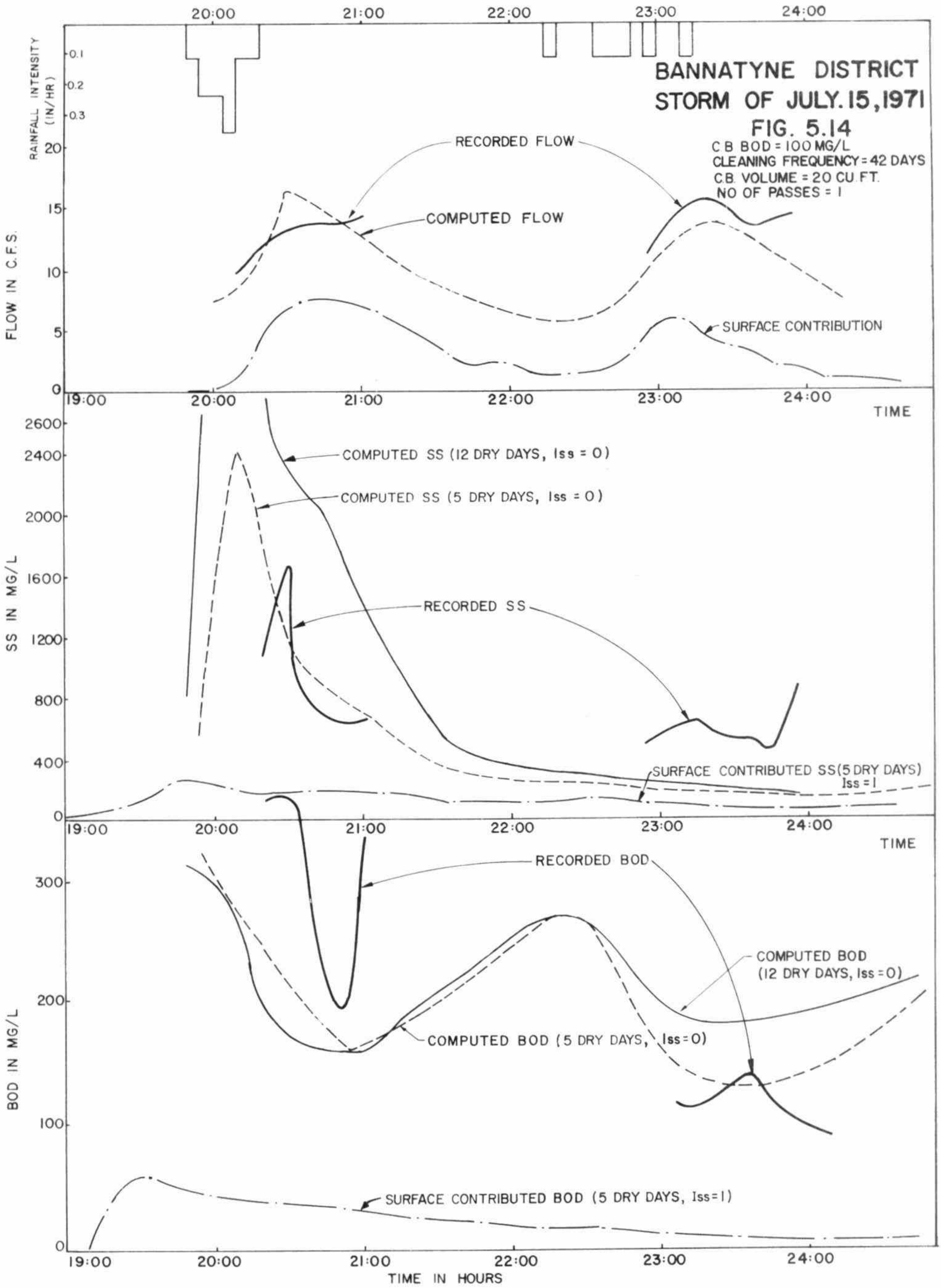
FLOW AND POLLUTOGRAPHS FOR BRUCEWOOD STORM OF AUG. 29, 1975
FIG. 5.11
RAINFALL INTENSITY (IN./HR.)
NO OF DRY DAYS = 2
CLEANING FREQUENCY = 14 DAYS
C.B VOLUME = 20 CU. FT.
C.B BOD = 60 MG/L
NO. OF PASSES = 1
DD = 0·5 LBS/DAY/100 FT. OF CURB
SUSPENDED SOLIDS = 830 MG/GM OF D/D
RECORDED FLOW
COMPUTED FLOW
FLOW IN CFS
COMPUTED SS Iss = 1
(USING MEASURED DATA FOR D/D AND SS)
RECORDED SS
COMUTED SS (Iss = 0)
SUSPENDED SOLIDS IN MG/L
COMPUTED BOD (USING MEASURED DATA FOR D/D AND SS Iss = 1)
COMPUTED BOD (Iss = 0)
RECORDED BOD
BOD IN MG/L
TIME IN MINUTES

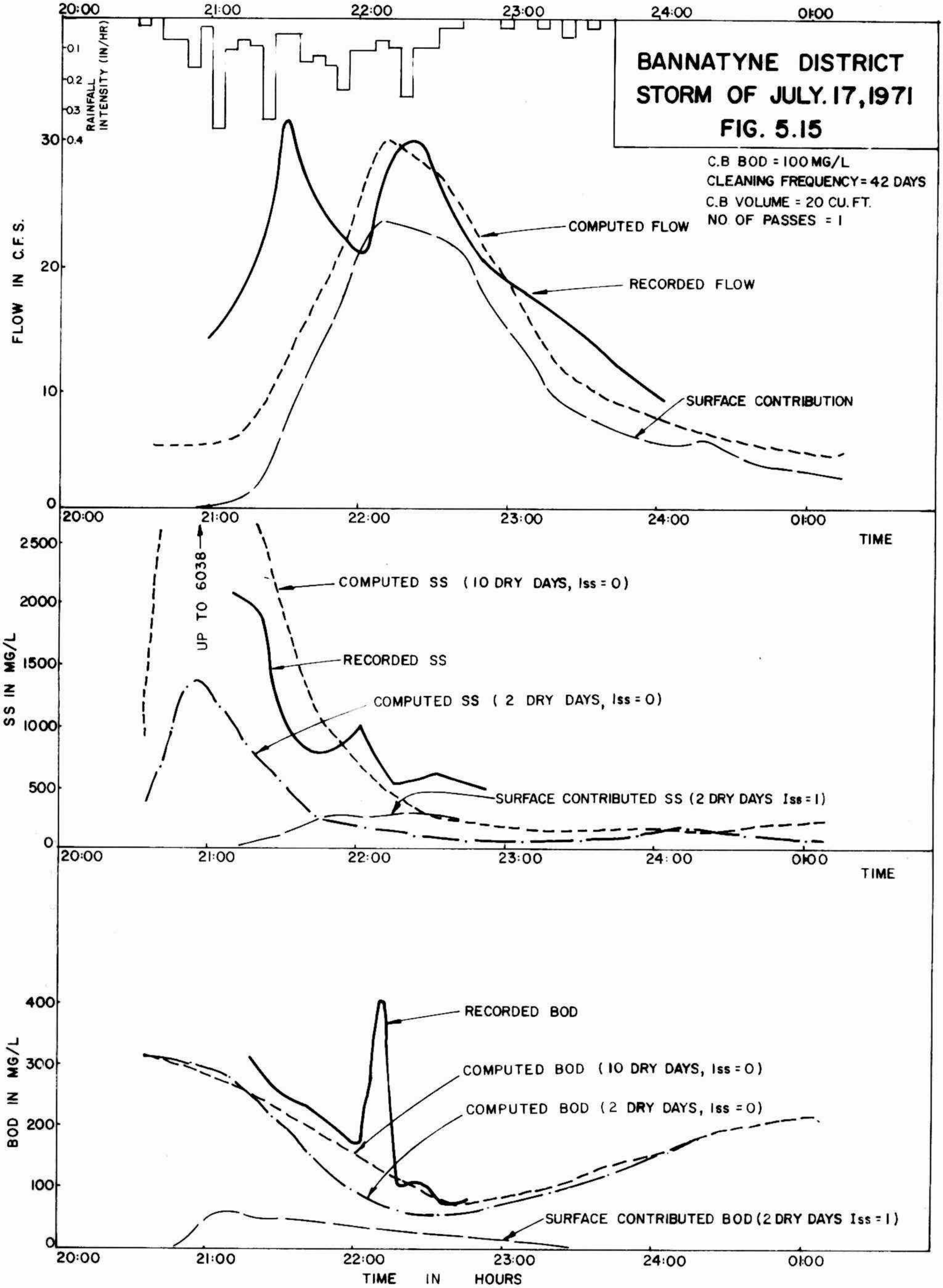


FLOW AND POLLUTOGRAPHS FOR BRUCEWOOD STORM OF SEPT. 11, 1975
FIG. 5.12
NO OF DRY DAYS = 12
CLEANING FREQUENCY = 14 DAYS
C.B. BOD = 60 MG/L
C.B. VOLUME = 20 CU. FT.
NO OF PASSES = 1
FLOW IN CFS
RAINFALL INTENSITY (IN/HR)
COMPUTED FLOW
RECORDED FLOW
SUSPENDED SOLIDS IN MG/L
COMPUTED SS (Iss = 0)
COMPUTED SS (Iss = 1)
RECORDED SS
BOD IN MG/L
COMPUTED BOD (Iss = 0)
COMPUTED BOD (Iss = 1)
RECORDED BOD
COLIFORM IN MPN/100 ML
RECORDED COLIFORM
CALCULATED WITH 0.65 x 10^6
TIME IN MINUTES

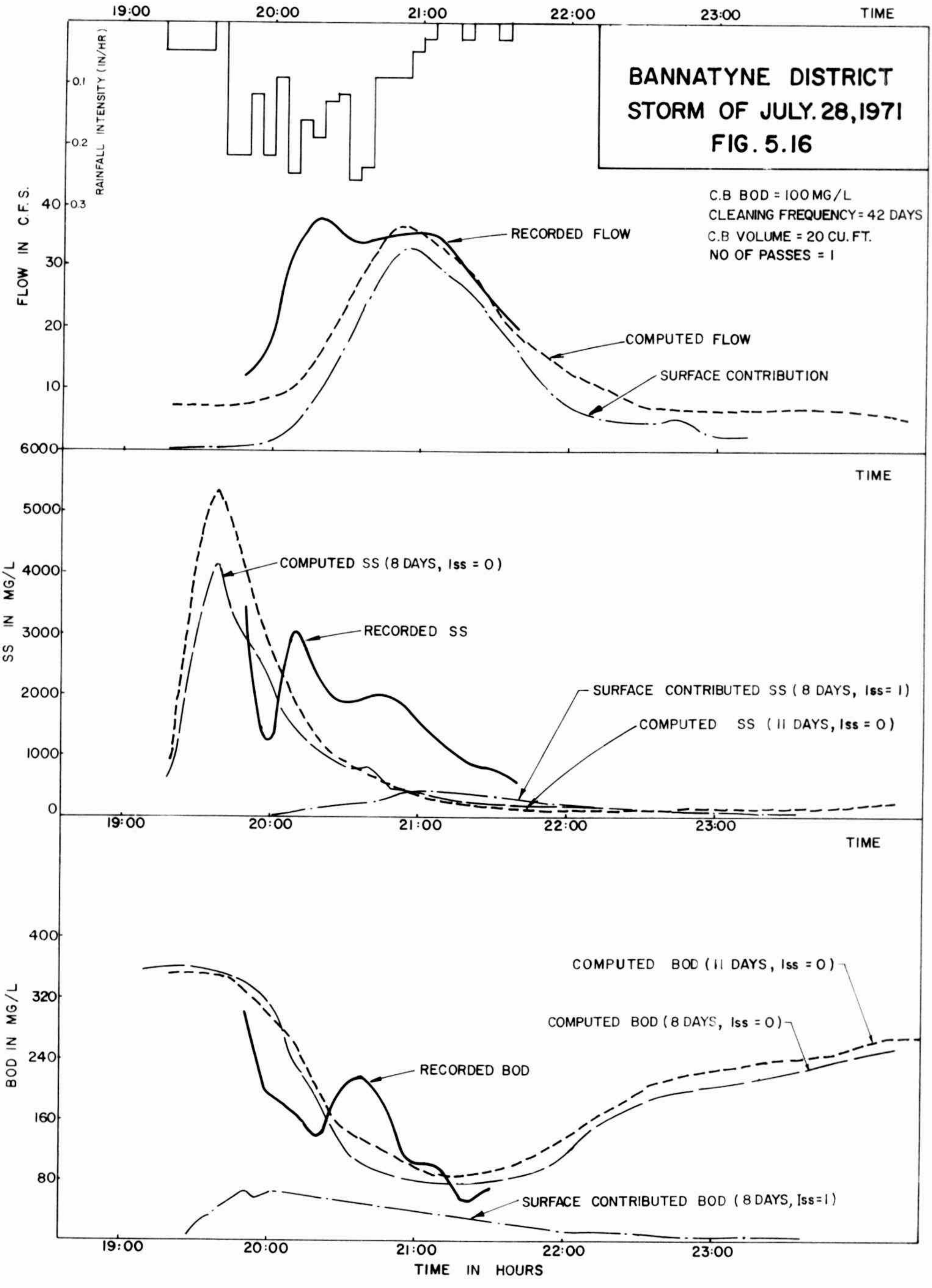


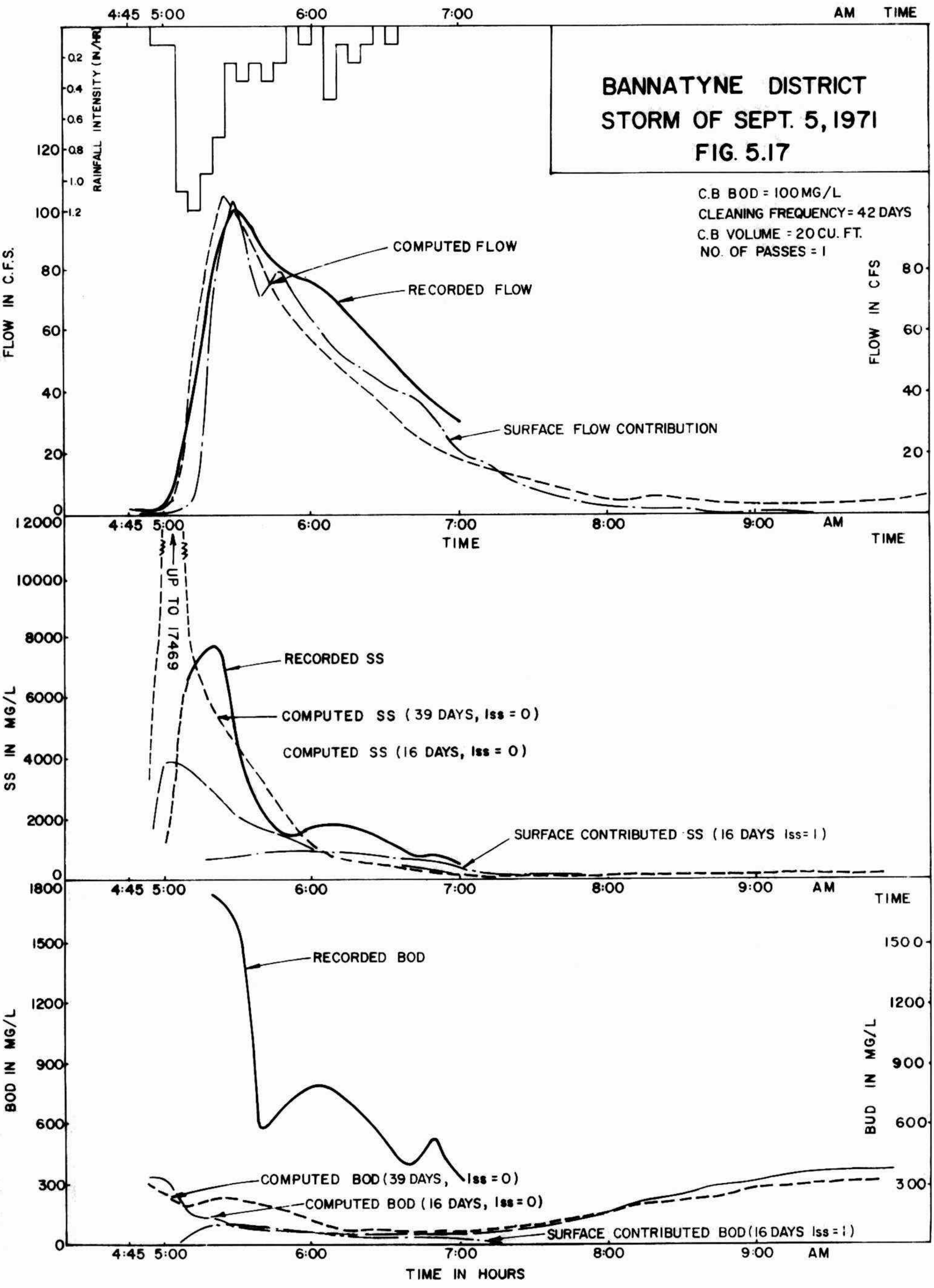
BANNATYNE DISTRICT
STORM OF JUNE 19, 1971
FIG. 5.13
C.B. BOD = 100 MG/L
CLEANING FREQUENCY = 42 DAYS
C.B. VOLUME = 20 CU. FT.
NO OF PASSES = 1
10:00
11:00
12:00
RAINFALL INTENSITY (IN/HR)
0·1
0·2
0·3
COMPUTED FLOW
RECORDED FLOW
SURFACE CONTRIBUTION ONLY
FLOW IN C.F.S
40
20
0
11:00
12:00
13:00
14:00
15:00
TIME
SS IN MG/L
3000
2000
1000
0
TO 4311
COMPUTED SS (8 DRY DAYS Iss=0)
RECORDED SS
COMPUTED SS (11 DRY DAYS Iss=0)
COMPUTED SS (8 DRY DAYS Iss=1)
SURFACE WASHOFF ONLY
10:00
11:00
TIME
12:00
BOD IN MG/L
320
240
160
80
0
COMPUTED BOD (11 DRY DAYS Iss=0)
COMPUTED BOD (8 DRY DAYS Iss=0)
RECORDED BOD
COMPUTED BOD (8 DRY DAYS Iss=1)
SURFACE WASHOFF ONLY
10:00
11:00
12:00
TIME IN HOURS


BANNATYNE DISTRICT
STORM OF JULY. 15, 1971
FIG. 5.14
C.B. BOD = 100 MG/L
CLEANING FREQUENCY = 42 DAYS
C.B. VOLUME = 20 CU. FT.
NO OF PASSES = 1
RAINFALL INTENSITY (IN/HR)
0.1
0.2
0.3
FLOW IN C.F.S.
RECORDED FLOW
COMPUTED FLOW
SURFACE CONTRIBUTION
SS IN MG/L
COMPUTED SS (12 DRY DAYS, Iss = 0)
COMPUTED SS (5 DRY DAYS, Iss = 0)
RECORDED SS
SURFACE CONTRIBUTED SS (5 DRY DAYS) Iss = 1
BOD IN MG/L
RECORDED BOD
COMPUTED BOD (12 DRY DAYS, Iss = 0)
COMPUTED BOD (5 DRY DAYS, Iss = 0)
SURFACE CONTRIBUTED BOD (5 DRY DAYS, Iss = 1)
TIME
19:00
20:00
21:00
22:00
23:00
24:00
TIME IN HOURS



BANNATYNE DISTRICT
STORM OF JULY. 17, 1971
FIG. 5.15


BANNATYNE DISTRICT
STORM OF JULY. 28, 1971
FIG. 5.16
C.B BOD = 100 MG/L
CLEANING FREQUENCY = 42 DAYS
C.B VOLUME = 20 CU. FT.
NO OF PASSES = 1
TIME
19:00
20:00
21:00
22:00
23:00
RAINFALL INTENSITY (IN/HR)
0.1
0.2
0.3
FLOW IN C.F.S.
40
30
20
10
RECORDED FLOW
COMPUTED FLOW
SURFACE CONTRIBUTION
SS IN MG/L
6000
5000
4000
3000
2000
1000
0
COMPUTED SS (8 DAYS, Iss = 0)
RECORDED SS
SURFACE CONTRIBUTED SS (8 DAYS, Iss = 1)
COMPUTED SS (11 DAYS, Iss = 0)
BOD IN MG/L
400
320
240
160
80
COMPUTED BOD (11 DAYS, Iss = 0)
COMPUTED BOD (8 DAYS, Iss = 0)
RECORDED BOD
SURFACE CONTRIBUTED BOD (8 DAYS, Iss=1)
TIME IN HOURS



BANNATYNE DISTRICT
STORM OF SEPT. 5, 1971
FIG. 5.17
C.B BOD = 100 MG/L
CLEANING FREQUENCY = 42 DAYS
C.B VOLUME = 20 CU. FT.
NO. OF PASSES = 1
4:45 5:00 6:00 7:00 AM TIME
RAINFALL INTENSITY (IN/HR)
0.2 0.4 0.6 0.8 1.0 1.2
FLOW IN C.F.S.
0 20 40 60 80 100 120
FLOW IN CFS
20 40 60 80
COMPUTED FLOW
RECORDED FLOW
SURFACE FLOW CONTRIBUTION
4:45 5:00 6:00 7:00 8:00 9:00 AM
TIME
SS IN MG/L
0 2000 4000 6000 8000 10000 12000
UP TO 17469
RECORDED SS
COMPUTED SS (39 DAYS, Iss = 0)
COMPUTED SS (16 DAYS, Iss = 0)
SURFACE CONTRIBUTED SS (16 DAYS Iss = 1)
4:45 5:00 6:00 7:00 8:00 9:00 AM
TIME
BOD IN MG/L
0 300 600 900 1200 1500 1800
BUD IN MG/L
300 600 900 1200 1500
RECORDED BOD
COMPUTED BOD (39 DAYS, Iss = 0)
COMPUTED BOD (16 DAYS, Iss = 0)
SURFACE CONTRIBUTED BOD (16 DAYS Iss = 1)
4:45 5:00 6:00 7:00 8:00 9:00 AM
TIME IN HOURS

ASSESSMENT OF THE SWMM INFILTRATION AND RECEIVING WATER BODY ROUTINES

CHAPTER 6

CHAPTER 6

ASSESSMENT OF THE SWMM INFILTRATION AND RECEIVING WATER BODY ROUTINES

6.1 GENERAL

The terms of reference of this study require an assessment of two additional SWMM routines INFIL and RECEIV. It was not possible to assess the INFIL routine due to data limitations. RECEIV was assessed only on the basis of a literature review, following a small number of simulations.

The INFIL subroutine which forms part of the TRANSPORT Block allows the user to model flows due to infiltration into the main conduits in a sewer system. Existing data indicate that rates of infiltration are highly site specific and therefore local measurements may often be required. If the importance of infiltration during particular circumstances, such as a high storm intensity is minimal, then a crude overall estimation of infiltration may be sufficient for modelling purposes. In other cases a more rigorous investigation possibly involving measuring programmes will be necessary.

The Receiving Water Model allows for the simulation of quantity and quality effects in rivers and estuaries as well as in unstratified lakes and reservoirs. Thus, the effects of stormwater discharges and combined sewer overflows on the receiving water may be broadly assessed. A versatile quality subroutine enables the improvements resulting from different combinations of storage and treatment to be rapidly investigated. Consequently, receiving water body simulations constitute an important factor in cost-benefit analyses for various stormwater management alternatives.

6.2 INFILTRATION

Flows resulting from infiltration and inflow reduce the capacity of sewer systems and treatment facilities to handle domestic and industrial waste waters. Frequently infiltration/inflow increase water pollution, causing

health hazards when untreated waste waters overflow directly to the receiving water. Infiltration may occur due to defective pipes, through pipe joints, connections or poorly maintained manhole walls. Inflow, which is a separate phenomenon, may enter the system via roof leaders, cellar, yard and area drains, foundation drains and cross connections. Additional sources may include catchbasins, manhole covers, swamp drainage and storm and wash waters. Inflow is accounted for in the RUNOFF Block, while the infiltration is determined separately in the INFIL routine, which forms part of the TRANSPORT Block.

Where appropriate flows resulting from miscellaneous sources, comprising the base dry weather, infiltration, antecedent precipitation, frozen residual moisture and high groundwater should be accounted for. Initial work carried out by Metcalf and Eddy [1] indicated that it is not possible to develop a widely applicable general predictive equation for infiltration since particular localized conditions exert a predominant influence on this process. Thus the availability of local data describing the contribution of the infiltration to measured flows should take precedence over generalized estimates. The INFIL subroutine may be used in conjunction with local data or if this is limited, with an overall estimate of the infiltration rate together with information on the total pipe length, diameter and number of joints. The contribution of antecedent precipitation to infiltration is assumed to depend on the magnitude of rainfall and the time elapsed since that rainfall. Local data can be used to generate coefficients for a linear regression equation describing infiltration due to antecedent rainfall:

$$RINFIL = ALF + ALF0 * RN0 + ALF1 * RN1 + \ldots\ldots + ALF9 * RN9 \qquad (6.1)$$

where ALFN = coefficients determined from regression analysis for N = 1, 2 ... 9 days antecedent precipitation

RNN = precipitation on N days prior to estimate (ins)

Nine days is considered to be the period, after which infiltration resulting from previous storms is insignificant. Residual moisture in the form of snow or ice is considered to contribute to infiltration when the degree day index exceeds 750. The model adds infiltration from this source where appropriate according to a sinusoidal relationship over the melt period.

In a situation where the groundwater table is above the pipe invert, groundwater infiltration is assumed to supercede infiltration from other sources. Estimation of the magnitude of the groundwater contribution is then based on a regression expression involving the groundwater head above the sewer invert and measured infiltration rates. In the absence of significant data, this source has to be assessed in a less rigorous manner (i.e. by calibration).

Data from several Canadian studies were collected and recommendations from the literature reviewed. There was a very wide variation of measured and design infiltration rates (Table 6.1). These data were not suitable for establishing regression equations similar to those outlined above since the typically very high values indicate the predominance of inflow rather than infiltration. Infiltration in combined systems is not normally significant when compared with greater flow volumes associated with storm runoff. Consequently, for combined systems infiltration can normally be ignored, or the phenomenon could be included as a constant rate. Since sanitary sewers with extraneous flows are operating as partially combined sewers, mathematical models such as SWMM, which simulate both dry weather flow and stormwater runoff may be important tools for assessment of system performance. Flow measurements are required to enable accurate calibration of the SWMM model to the sanitary system under study. Calibration may be achieved by adjusting areas with contributing runoff until good agreement is achieved between predicted and recorded flows. Design storms may be simulated with the calibrated model to identify specific sanitary sewer back-up and other system inadequacies and to test the effectiveness of proposed relief measures.

A modelling approach was used in San Francisco to identify sources of infiltration/inflow, to assess performance of existing sanitary collection and treatment systems, to determine overflow characteristics, and to develop cost-effective alternatives for system control [2].

TABLE 6.1

SOME MEASURED AND DESIGN INFILTRATION RATES

Source	Measurement	Design	Infiltration Gallons Acre/Day
Gore and Storrie, York-Pickering Sewer Project, Ont.		X	1290
J.F. MacLaren Ltd. Halifax, N.S.	X		163-3446
Proctor & Redfern Ltd. Sault Ste. Marie, Ontario	X	X	462-1990
J.F. MacLaren Ltd. Atlantic Prov. Studies			25000 avg. 2500
Canadian British Ltd. Halifax, N.S.	X		10,000-40,000
Phillips & Roberts Ltd. Burlington, Ontario	X		720-2240
Proctor & Redfern Ltd.		X	1290
J.F. MacLaren Ltd. Burlington, Ontario		X	2200
J.F. MacLaren Ltd. Ottawa, Ontario		X	3000
Black & Veatch Mission Twp., Kansas	X		10,000

6.3 RECEIVING WATER BODY MODEL

In order to assess the need for and the efficacy of various stormwater management and pollution abatement schemes, it is necessary to study the effects of pollutant discharges in the receiving water.

Pollutant concentrations resulting from an untreated stormwater discharge or a combined sewer overflow must be known or predicted so that any deletereous effects can be quantified, and compared with standards or regulations. If violations are evident the water quality improvements resulting from various storage and treatment policies should be investigated and related to the costs of treatment. Such a procedure enables planners and decision makers to determine the cost effectiveness of various management schemes.

6.3.1 *General Discussion of Receiving Water Quality Models*

Traditionally water quality models have been limited to the solution of the dissolved oxygen balance in rivers subject to constant waste discharges. The earliest models gave only steady state solutions for the dissolved oxygen (D.O.) profile along the river and served to identify the location of the maximum D.O. sag [3]. More recently, dynamic models have been used to define the temporal and spatial variations in the D.O. along the river, such as in the Thames River and Saint John River Basin studies in Canada [4,5]. Estuarial systems are the most complex aquatic systems and consequently the procedures for modelling these are generally more complex than for simple nontidal river systems. One of the most extensively applied estuarial models is that of the Delaware River Estuary [6]. This was proposed as a tool to describe the seasonal steady state D.O. concentrations resulting from alternative pollution control policies. The model requires the system to be divided into a series of perfectly mixed finite segments within which the flow is uni-directional. The original model has been expanded to include the prediction of the fate of nitrogens in the estuary [7]. The model has also been applied for the Potomac River [8] and for Hillsborough Bay in Florida [9]. The M.I.T. one-dimensional dynamic network model was applied in a study of the St. Lawrence River [10].

The SWMM Receiving Water body model is a particularly versatile dynamic simulation model. In the RECEIV subroutine of the SWMM, the receiving water is discretized by dividing the system into a series of one-dimensional channels. Where the actual channel broadens into an embayment or estuary, the system may be represented by a network of one-dimensional elements, connecting nodal points, approximating the volumetric and flow effects in the natural system. Thus a continuous area of water such as an estuary or lake may be simulated.

The RECEIV subroutine requires the input of the average channel dimensions and roughness as well as nodal characteristics including surface area, depth and surface elevation. The surface area of a node in a simple river simulation is the sum of the two adjacent channel half areas. In a more complicated triangular network, the nodal areas are determined manually by the Thiessen polygon method or the model subroutine TRIAN may be used to generate approximate junction areas and channel dimensions based upon a minimum of input data. The hydrodynamic information generated in the QUANTITY subroutine forms the basis for QUALITY simulations. Variable water withdrawals and additions, waste loading inputs and a variety of tidal and nontidal boundary conditions may be modelled and the distribution of conservative and non-conservative constituents simulated. An explicit solution to the equations of continuity and momentum is applied at each time step. Constituent concentrations are established from the conservation of mass equation.

Earlier versions of the model were applied by WRE for the simulation of water quality in the Sacremento-San Joaquin Delta [11] and for the investigation of potential problems associated with landfilling in the San Francisco Bay. The SWMM and associated models have proved to operate well in the simulation of the mass transport of conservative substances in situations where advection is the major transport mechanism [12,13]. One of the limitations of the SWMM approach is the absence of a diffusion term in the mass transport equations.

This was emphasized in a recent application of SWMM to a network of Finger Fill Canals in Florida [14]. For this particular configuration the model failed to correctly predict the mass transport in canal systems; the simulated transport of a dye cloud lagged far behind the observed movement. The authors also applied the Columbia River Model [15,16], which like SWMM, was developed from the original WRE node and channel model. However, this model includes a diffusion term in the transport equations. The results indicated that "in situations where the net advective transport is small, the SWMM is not capable of simulating mass transport". The Columbia River model, with an appropriate diffusion constant was shown to adequately simulate mass transfer in these types of systems.

A good example of a sophisticated two dimensional receiving water model is that developed by the Rand Corporation for the simulation of coastal and estuarine systems [17]. This model solves the basic differential equations of motion and continuity by a time centered difference scheme and an integrated quality simulation routine based on the mass balance equation that solves the quality equations by an alternative direction implicit-explicit technique. This model, which uses a grid system to represent the water body, has been applied for Jamaica Bay, New York City [18]. The model was modified and applied for the simulation of water quality in Hamilton Harbour, Ontario [19].

Another two-dimensional simulation model was developed originally for the U.S. Atomic Energy Commission for predicting the behaviour of thermal plumes [20]. This model (HYETA), which uses a horizontal square grid system of discretization similar to the Rand model, has been adapted for the simulation of water quality constituents. HYETA accounts for advective and diffusive transport. A comparison of the results of SWMM and HYETA for a hypothetical receiving water is discussed in the next section.

The computational costs and data requirements of different models must be balanced against the degree of accuracy required, which is usually determined by the scale and stage of development (i.e. screening, planning or design) of the project.

6.3.2 *Applications of the Model*

Several test applications were carried out in order to test the response of the RECEIV Block. Figure 6-1 shows the computed suspended solids profiles along a simplified river system. The movement of the concentration peak downstream is simply related to the average flow rate (i.e. there does not appear to be significant "numerical dispersion"). A typical lakefront sewer outfall is shown in Figure 6-2 in two alternative locations (inside and outside the breakwater). The changes in the level of pollution of the receiving water resulting from the relocation of the sewer outfall to discharge outside the breakwater were investigated. Hypothetical pollutographs based on typical recorded stormwater pollutant concentrations were used in these simulations. The plots shown on Figure 6-3 indicate the coliform concentrations at the nodes inside and outside the breakwater for each alternative. A coliform decay rate coefficient of 2 day^{-1} was used. The plots show rather qualitatively that discharge into the deeper waters outside the breakwater utilizing the greater dilution available is preferable to discharge inside the breakwater (assuming a water quality standard of about 10^3 MPN/100 ml).

For future applications of the model in practical situations it will be necessary to assess the performance of the model compared with measurements and with the results of more sophisticated models. It was possible to compare the SWMM and the HYETA models within the scope of the present study. The comparison was based upon a hypothetical situation in which a river highly polluted by suspended solids (200 mg/l) flows into a semi-enclosed embayment

with a small harbour at one side of the river mouth (Figure 6-3). The depth varies from 3 feet at the river mouth to 6 feet at the open end of the system. The distribution of suspended solids in the system predicted by the two models after 12 hours of simulation is indicated on Figure 6-3. While the two simulations are reasonably in agreement, two dissimilarities are noteworthy: SWMM appears to cause more pollutant to be advected into the area behind the breakwater, while HYETA predicts a more widespread dispersion throughout the whole area. Since verification of these findings is not possible, definite conclusions cannot be drawn. Although the fundamental representation of the flow field and the inclusion of the diffusive term in the HYETA model is probably more accurate, the results of the SWMM indicate that it may be used for comparison of alternatives at a screening of planning stage.

6.3.3 *Remarks Concerning the RECEIV Model*

Several ommissions in the Users' Manual [21] became apparent during the initial applications of the model:

(a) The initial time for start of hydrograph input from cards (TZERO) is controlled by the day cycle on which printed output begins. A discharge hydrograph will not be read until there is 1 day cycle of printed output. This is not stated in the input description.

(b) It is necessary to have at least one stage v. time plot when using the tidal boundary condition. This can be for any node in the network. (i.e. NPLT$\geq$1).

(c) The model computes the decay of any non conservative constituents as a first order process; analogous to the biological oxidation of BOD:

$$\frac{dC}{dt} = \text{DECAY} * C \qquad (6.2)$$

where C = the concentration of the constituent at time t

DECAY = the first order rate constant

Any non conservative pollutant is assigned an equivalent conservative constituent by the model. This is analogous to the adding of Dissolved Oxygen (DO) when BOD is modelled. Of course when there is zero DO the BOD ceases to decay. DO is replaced by reaeration which is controlled by the rate constant REAER in the model.

In order to ensure that non conservative pollutants (such as coliforms), other than BOD can decay, it is necessary to ensure that there is a sufficient supply of the equivalent conservative constituents (analogous to DO). The most effective method for ensuring this was found to be the specification of a value for REAER for the non conservative constituent equal to the decay constant, DECAY.

These findings were discussed with representatives of the University of Florida and it was understood that subsequent editions of the Users' Manual will be modified accordingly.

The minimum time step possible for a stable solution in the QUANTITY subroutine is given by the Courant condition.

$$\Delta t = \frac{\Delta x}{\sqrt{gh}} \qquad (6.3)$$

Hence the minimum value of $\Delta x / \sqrt{gh}$ determines the time step for hydraulic computations. However, it is normal to reduce this time step by multiplying by a safety factor of 0.75 to ensure that the solution remains stable. A time step of 25 seconds was used in the lake simulation described in the previous section. The tidal boundary condition is the most appropriate when modelling only part of a large body of water. In the lake example, a tidal fluctuation with zero amplitude proved to be the most flexible boundary condition.

6.3.3 *Conclusions*

(a) The Receiving Water Body model of the SWMM does not include a diffusion term and should not be applied in circumstances in which advection is not the major transport mechanism.

(b) The testing of the model and limited comparison with more sophisticated models indicate that when, carefully applied, the SWMM may be used as a tool for the assessment of the relative effects of various pollution abatement measures at a planning stage.

(c) Several additional user requirements were identified and these are being included in updated versions of the model.

REFERENCE - CHAPTER 6

1. Metcalf and Eddy Inc. et al, "Stormwater Management Model - Vol. 1 - Final Report" for EPA Contract No. 14-12-501, July 1971

2. "The Control of Wet Weather Overflows and Bypasses" East Bay Municipal Utility District, Special District One (San Francisco) - Infiltration/ Inflow Analysis, a report by Water Resources Engineers Inc. February 1975.

3. Streeter, H.W., and Phelps, E.B., "A Study of the Pollution and Natural Purification of the Ohio River". III Bulletin U.S. Public Health Service No. 146.

4. "Thames River Basin Study", a report prepared by the Ontario Ministry of the Environment, WQB 1975.

5. "Water Quality Management Methodology and its Application to the Saint John River Study" a report to the Ministry of the Environment, Policy and Planning Branch, by H.G. Acres Ltd., 1971.

6. Federal Water Pollution Control Administration, "Delaware Estuary - A Comprehensive Study -- Preliminary Report and Findings", Department of the Interior, Philadelphia, Pa. 1966.

7. Thomann, R.V., "Systems Analysis and Water Quality Management", Environmental Science Service Publishing Co., Stanford, 1970.

8. Hetling, L.J., The Potomac Estuary Mathematical Model, Tech. Report, Cheasapeake Technical Support Laboratory, FWQA, Annapolis, Md.

9. Federal Water Quality Administration, "Problems and Management of Water Quality in Hillsborough Bay, Florida," Hillsborough Technical Assistance Project, Southwest Region, FWQA, 1969.

10. Thatcher, M.L. et al. "Application of a Dynamic Network Model to Hydraulic and Water Quality Studies of The St. Lawrence River" Proc. 2nd Annual Symp of the Waterways, Harbours and Coastal Eng. Div. ASCE, 1975.

11. Central Pacific Basins Comprehensive Water Pollution Control Program "Effects of the San Joaquin Master Drain on Water Quality of the San Francisco Bay and Delta", U.S. Department of the Interior, Federal Water Pollution Control Administration, Southwest Region, San Francisco, Calif., January 1967, 101 pp.

12. Kaiser Engineers, et al, "San Francisco Bay-Delta Water Quality Control Program", Final Report to the State of California Water Resources Control Board, June 1969.

13. Feigner, K.D., H.S. Harris, "Documentation Report FWQA Dynamic Estuary Model," U.S. Dept. of the Interior, FWQA, July 1970, 248 pp.

14. Barnwell, T.O., Cavinder, T.R., "Application of Water Quality Models to Finger Fill Canals" Proc. 2nd Annual Sym of the Waterways, Harbours and Coastal Eng. Div. ASCE 1975.

15. Callaway, R.J., R.V. Byran and G.R. Ditsworth - Mathematical Model of the Columbia River from the Pacific Ocean to Bonneville Dam. Part 1 - Theory, Program notes and Programs. USDI, FWQA Pacific Northwest Water Laboratory, Corvallis, Oregon, November 1967, N.T.IS No. PB 20-2-422

16. Callaway, R.J. and K.V. Byran - Mathematical Model of the Columbia River from the Pacific Ocean to Bonneville Dam - Part II - Input/Output and Initial verification Precedures. U.S. EPA, Pacific North West Water Lab., Corvallis, Oregon, December 1970 NTIS No. PB202-423

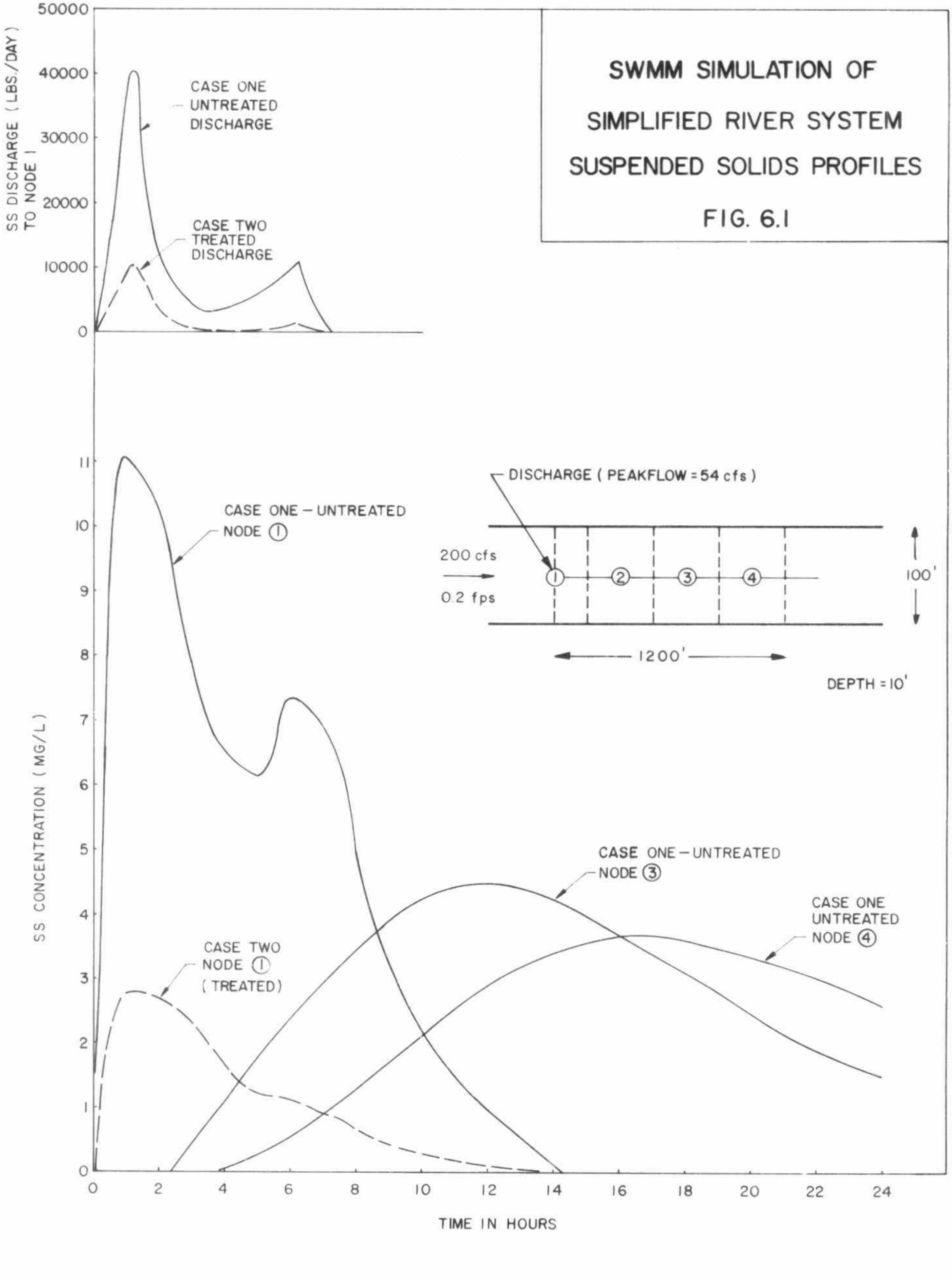
17. Leendertse, J.J., "Aspects of a Computational Model for Well-mixed Estuaries and Coastal Seas", RM 5294-PR, The Rand Corporation, Santa Monica, California 1967.

18. Leendertse, J.J. and E.C. Gritton, "A Water Quality Simulation Model for Well-mixed Estuaries and Coastal Seas", Vol. III, Jamaica Bay Simulation, R-709-NYC, The New York City Rand Institute, New York, July 1971.

19. "Hamilton Harbour Study" a report prepared by the Ontario Ministry of the Environment WQB, May 1975.

20. Edinger, J.E. et al. "Generic Emergency Cooling Pond Analysis" a report prepared for the U.S. Atomic Energy Commission under contract no. AT (11-1) - 2224, October 1972

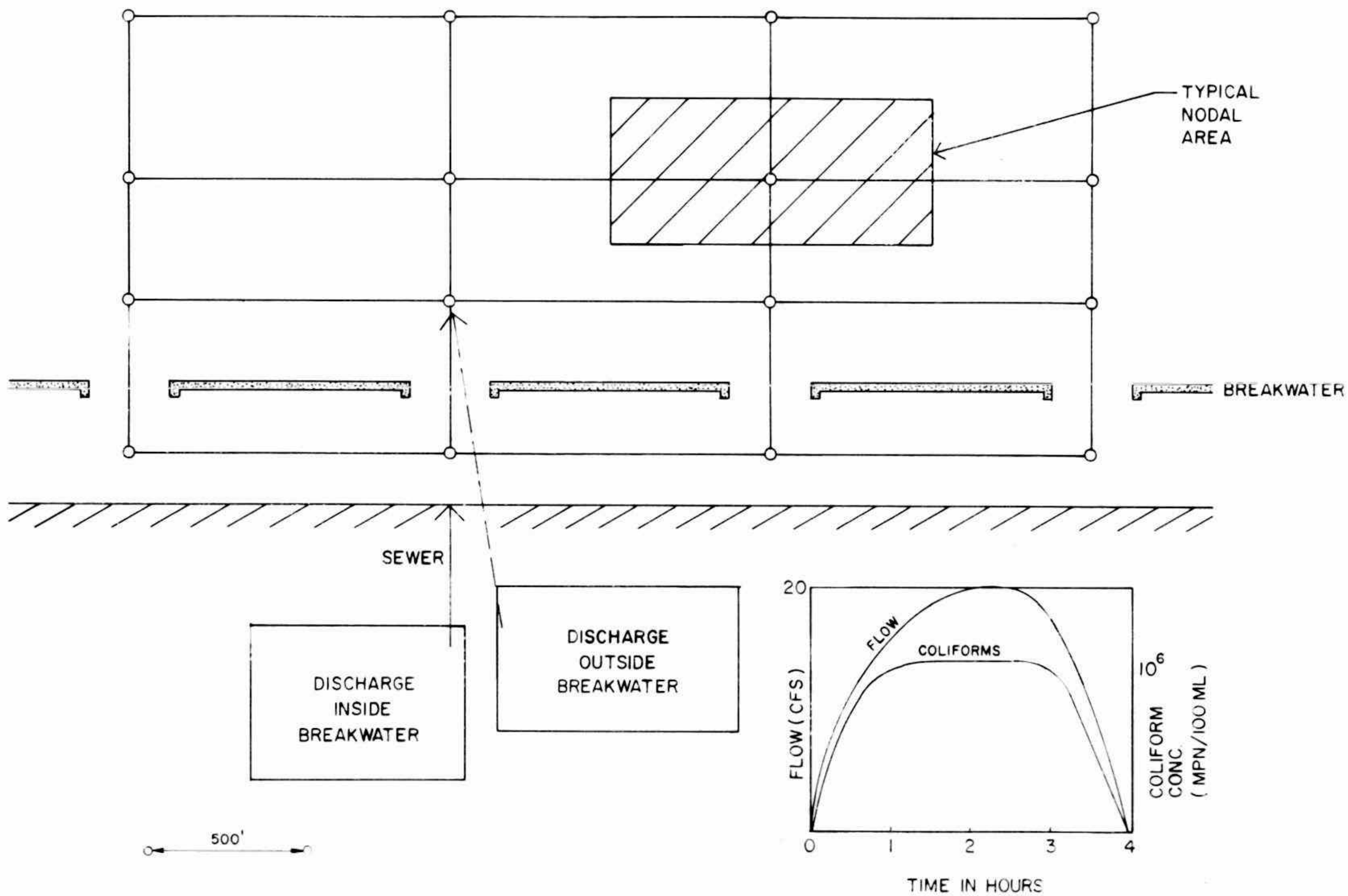
21. Huber, W.C. et al. "Storm Water Management Model - User's Manual, Version II" for EPA, EPA-670/2-75-017, March 1975.

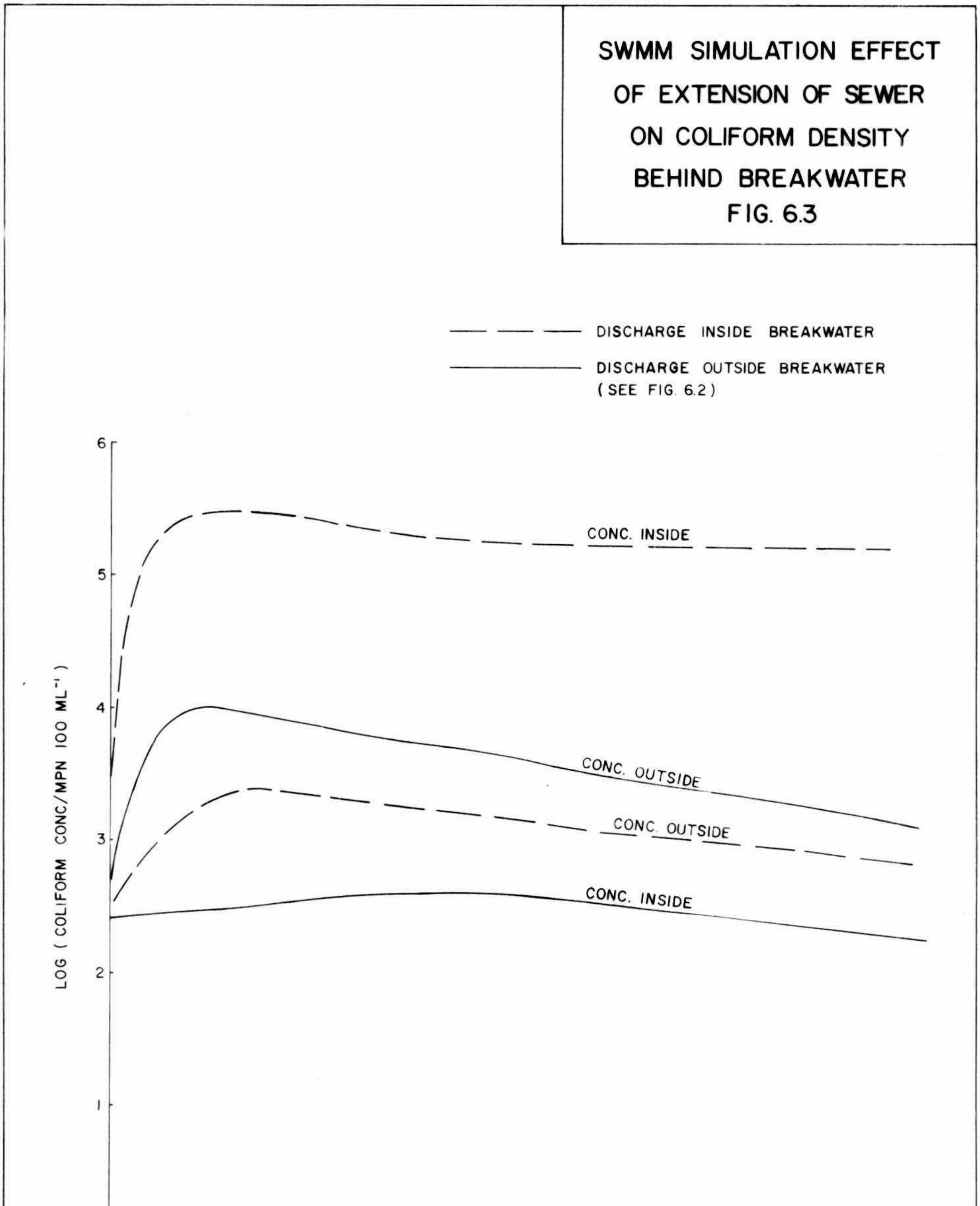

SWMM SIMULATION OF
SIMPLIFIED RIVER SYSTEM
SUSPENDED SOLIDS PROFILES
FIG. 6.1
SS DISCHARGE (LBS./DAY) TO NODE 1
50000
40000
30000
20000
10000
0
CASE ONE UNTREATED DISCHARGE
CASE TWO TREATED DISCHARGE
SS CONCENTRATION (MG/L)
CASE ONE - UNTREATED NODE 1
CASE TWO NODE 1 (TREATED)
CASE ONE - UNTREATED NODE 3
CASE ONE UNTREATED NODE 4
DISCHARGE (PEAKFLOW = 54 cfs)
200 cfs
0.2 fps
1
2
3
4
100'
1200'
DEPTH = 10'
0 2 4 6 8 10 12 14 16 18 20 22 24
TIME IN HOURS

LAKEFRONT SCHEMATIZATION

FOR SWMM SIMULATION

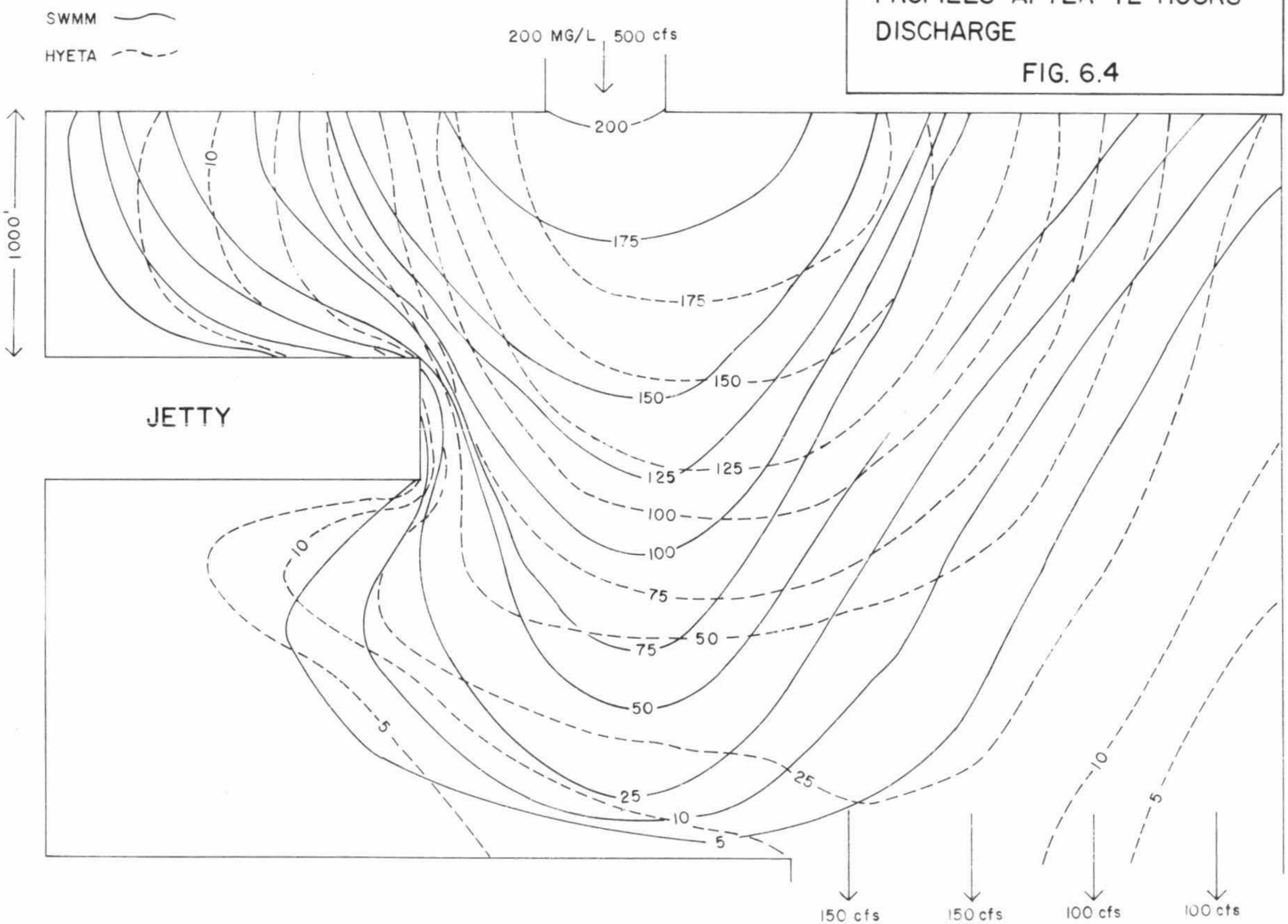
FIG. 6.2




SWMM SIMULATION EFFECT
OF EXTENSION OF SEWER
ON COLIFORM DENSITY
BEHIND BREAKWATER
FIG. 6.3
DISCHARGE INSIDE BREAKWATER
DISCHARGE OUTSIDE BREAKWATER
(SEE FIG. 6.2)
CONC. INSIDE
CONC. OUTSIDE
CONC. OUTSIDE
CONC. INSIDE
LOG (COLIFORM CONC/MPN 100 ML⁻¹)
6
5
4
3
2
1
0
4
8
12
16
20
24
STORM DISCHARGE
TIME IN HOURS

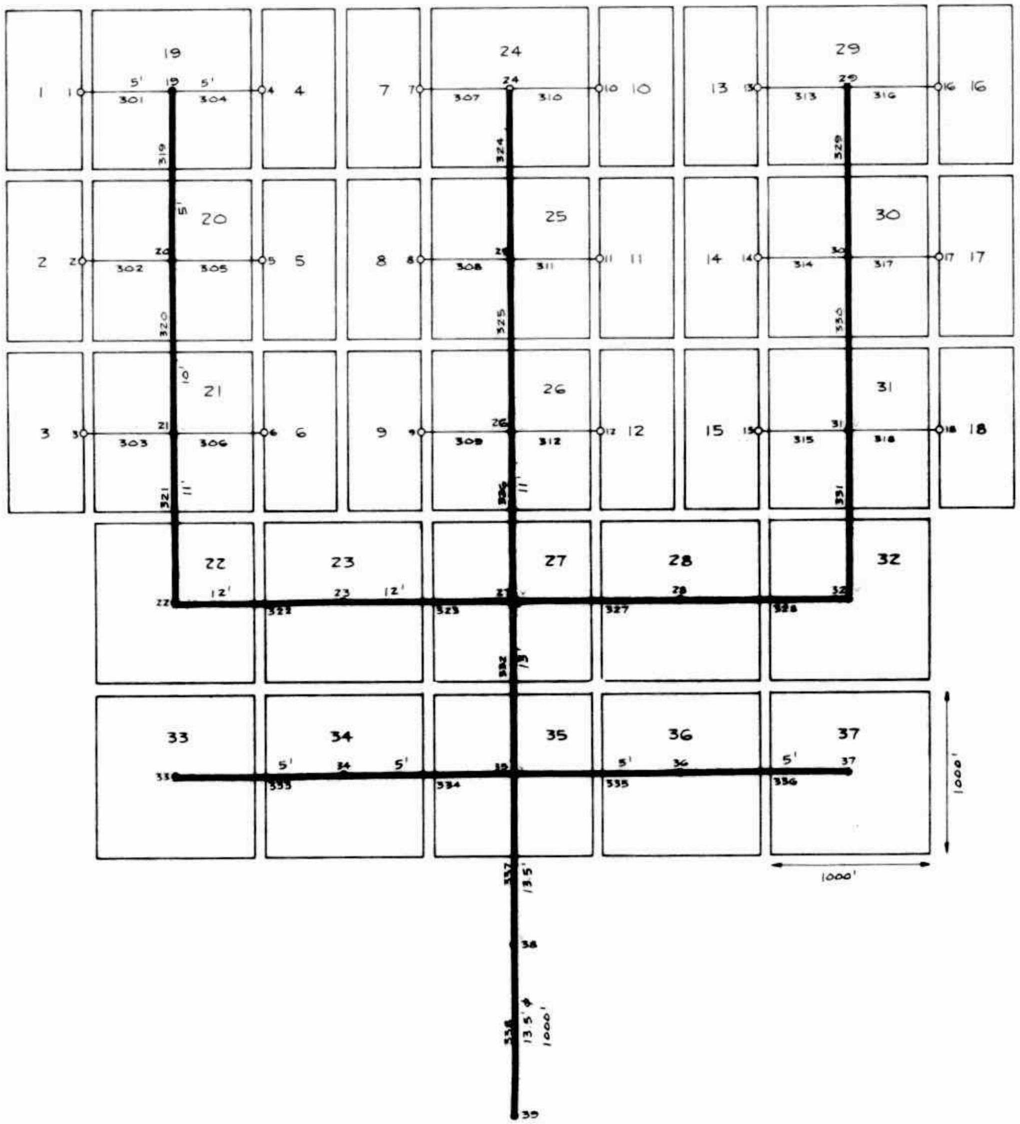
COMPARISON OF SWMM AND HYETA SUSPENDED SOLIDS PROFILES AFTER 12 HOURS DISCHARGE

FIG. 6.4

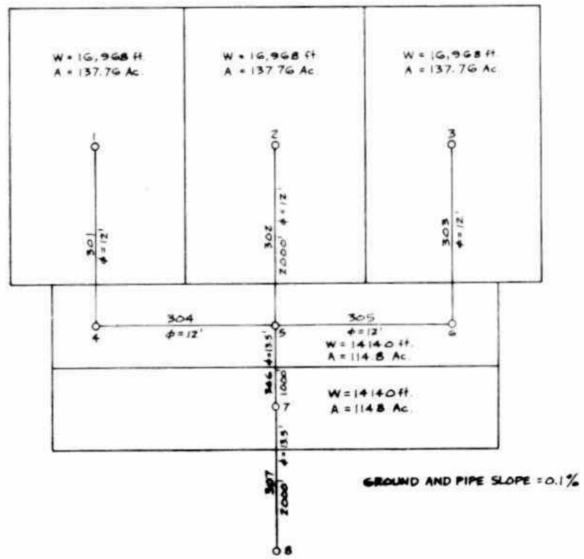


HYPOTHETICAL LUMPING TEST AREA LAYOUT

DETAILED DISCRETIZATION



HYPOTHETICAL TEST AREA LUMPED TO 5 SUBCATCHMENTS



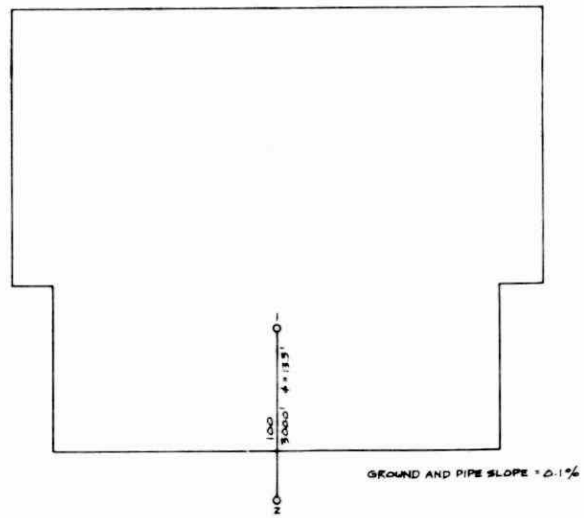


FIG. 6.5

HYPOTHETICAL TEST AREA LUMPED TO ONE SUBCATCHMENT

STORAGE/TREATMENT ROUTINES

CHAPTER 7

CHAPTER 7
STORAGE/TREATMENT ROUTINES

7.1 GENERAL

Equations describing the complex physical, chemical and biological processes associated with the various storage and treatment units are usually highly empirical and based upon limited data, consequently most available models are planning aids rather than design tools. A planning model should be capable of predicting the water quality improvements resulting from different levels of storage and treatment of a contaminated discharge and should relate a cost to different degrees of abatement. The decision maker should thus be able to select the optimal area for a detailed cost-benefit analysis and subsequent capital investment.

In this chapter the basic assumptions of the SWMM storage and treatment models are discussed and some inconsistencies are rectified. A cost estimation subroutine has been developed for Canadian conditions. The passage of a particular storm flow hydrograph and pollutograph is traced through a hypothetical treatment plant and a set of the results of the cost estimation routine are presented.

7.2 STORAGE/TREATMENT OPTIONS AND BASIC ASSUMPTIONS

7.2.1 *Storage Treatment Options*

Storage of discharges, together with the resulting sedimentation occurring during storage, with subsequent release into the treatment system appears to be the most cost effective method available for reducing stormwater related pollution [1]. Storage facilities may be constructed upstream of the treatment unit in order to provide flow equalization and dilution of the

typically high "first flush" pollutant concentrations. Such "in line" storage may be available in the sewer system, particularly in systems with flat sewer grades, and may be utilized by appropriate flow constriction upstream of overflow points, or may be provided immediately ahead of the treatment facilities. Alternatively, storage may be provided "off-line", as a means to retain contaminated flows until spare capacity for treatment becomes available. "Off-line" storage facilities may take the form of conventional sedimentation tanks, underground and void space storage, or underwater bag storage.

The treatment processes currently employed in the SWMM for reduction of pollution from storm sewer discharges and combined sewer overflows are briefly described in the following:-

(1) Bar Racks - The main function of bar racks is to trap and remove those large objects from the waste stream, which might subsequently damage downstream equipment. Bar racks are generally installed for all storage, treatment and pumping facilities. A typical bar layout provides one inch clear openings spaced at two inches between centres.

(2) Swirl Concentrator - The swirl concentrator is primarily employed for the rapid removal of settleable solids. Preliminary investigations indicate that this equipment is particularly appropriate for the treatment of storm flows, responding well to fluctuating flows [2]. The removal efficiency improves as the ratio of stormflow to dry weather flow decreases. The swirl concentrator may also be used at combined sewer outfalls to allow a large proportion of the solids to be intercepted and delivered to the interceptor system.

(3) Fine Screens - Fine screens were often considered as an alternative to sedimentation and have often been used in industrial applications.

The screen wire may vary from 6-mesh to 60-mesh depending on the characteristics of the waste; a 50-mesh screen is most common. Problems occur in operation when the screens, which may be rotary or drum types, become clogged with oil or grease and cannot be cleared by normal backwashing. Fine screens generally remove about 38 and 16 percent of suspended solids and BOD respectively, although the efficiency of a particular screen is dependent on the size of wire mesh and the buildup of smaller particles on the screen [3].

(4) Sedimentation - Sedimentation is the most established and widespread method of reduction of solids and associated BOD from waste waters. Conventional design requires detention times of one to two hours. The removal efficiency is dependent upon the residence time which may vary considerably during a storm, since in most cases the sedimentation process would take place in the storage facility. Removal efficiencies in conventional units are approximately 60 and 30 percent for suspended solids and BOD respectively for a residence time of 2 hours.

(5) Dissolved Air Flotation - Dissolved air flotation has several advantages over conventional sedimentation. High overflow rates may be used because of the greater effective difference in specific gravity between the water and combined (air bubble and particle) particle, than that in normal sedimentation. High overflow rates allow short detention times and the upward separation and subsequent skimming allows for removal of particles less dense than water in addition to surface oil and grease. The characteristics of the individual waste stream greatly affect achievable removal efficiencies and consequently pilot plant studies will usually be required before this process is installed.

(6) Microstrainers - This unit is similar to the drum screen (fine screen) device in operation. The drum rotates about a horizontal axis allowing gravity filtration, with continuous backwashing at the top of the drum. As with fine screens, the removal efficiencies are dependent upon the type of media used and the concentration and characteristics of the solids in the waste stream.

(7) High Rate Filters - Filtration removes finer particles than those removed in screening processes. In conventional plants filtration is usually regarded as a polishing treatment. High rate filters generally consist of dual media or mixed media. At present, operating costs are high and frequent backwashing is required to prevent clogging, but land requirements are not extensive and the process is easily automated.

(8) Biological Treatment - For successful biological treatment of storm related wastes, it is necessary to maintain an active microbial biomass to utilize the dissolved organic and non-organic matter in the waste flow. This may be achieved either by operating the wet weather unit in parallel with the conventional dry weather plant or by using the dry weather plant for dry and wet weather flows. The latter would be achieved using treatment units able to operate efficiently over a wide range of flows and substrate concentrations. Units such as trickling filters tend to operate well under conditions of fluctuating waste strength but upstream flow equalization is probably essential to prevent excessive hydraulic loads. In addition, all biological treatment processes necessitate downstream units for settling out of cell tissue, and subsequent sludge disposal.

(9) Effluent Screens - These screens serve only to remove floatable and unsightly material that might remain after low levels of

treatment. Their function is essentially aesthetic improvement of the discharge, there being no discernible associated improvement in water quality.

(10) High Rate Disinfection - Disinfection of waste discharges is generally achieved by chlorination. This results in a reduction in the coliform content of the waste in the order of 99.9%. The efficacy of the disinfection depends on the free chlorine residual concentration and the time of contact. High rate processes usually operate with a 15 minute detention period at the design flow.

7.2.2 *Discussion of Basic Assumptions*

The storage subroutine in SWMM, STORAG, allows for in-line and off-line storage basins. In-line storage implies all flows from the sewer system flow through the storage basin prior to treatment, whereas off-line storage is only utilized when the treatment capacity is exceeded. Two idealized flow regimes are allowed, these being plug flow and perfect mixing. Hydraulically the choice of regime in the model is not important, but when pollutant concentrations are of interest, the flow regime is of great importance. Plug flow is more appropriate for in-line storage and perfect mixing for off-line facilities.

The STORAG subroutine recycles flows from the storage basin that are in excess of the treatment capacity back to the storage basin. This recycling is not always practical; in general flows in excess of the treatment capacity are allowed to overflow once the capacity of the storage basin is reached. The removal of suspended solids in off-line storage basins is considered to occur subject to an exponential decay equation. The assumption of the completely mixed basin is not considered appropriate for stormwater storage facilities and sedimentation would probably be better approximated using a mass balance approach.

The treatment subroutine, TREAT includes all the treatment processes discussed in the previous section. The flow chart in Figure 7-1 indicates the possible sequential series of storage and treatment options. Not all of the "strings" of treatment processes are technically feasible or economically realistic. However, a safety mechanism is available in this subroutine whereby the program will terminate upon location of an unfeasible "string" of options and will identify economically unsound combinations with a warning message. The possible combinations at present available are indicated in Figure 7-2.

The cost of a specific combination of storage and treatment is a function of design flow rate. Unless this is specified by the user, the model assesses the peak flow rate from the inlet hydrograph and sizes the treatment units accordingly. An evaluation of the range of treatment efficiencies possible for a particular storm discharge is obtained by specification of a series of design flow rates.

The cost estimate for a particular combination of storage and treatment facilities is determined in the cost subroutine, TRCOST. The costs are a function of the design flow rate, as in the model the unit size determines the capital outlay required.

7.3 MODIFICATIONS

In the original storage subroutine flows released from the storage basin, controlled by weirs, constant speed pumps or orifices could either flow directly to the treatment facilities or to a by-pass. The model considered flows discharged from storage, that were in excess of the treatment capability, to be recirculated to the storage basin. This process was modified to cause flows in excess of the treatment capacity to be routed to the bypass stream.

The original model removed suspended solids in the storage basin according to an exponential decay equation based upon the assumption of perfect mixing. The removal process was altered to reflect a more empirical basis [4] and this was incorporated in an overall mass balance equation:

$$\begin{aligned}(\text{Inflow SS}) = (\text{Outflow SS}) &+ (\text{SS Removed in storage basin})^{*} \\ &+ (\text{SS in suspension in storage basin})\end{aligned} \quad (4.1)$$

This approach represents actual flow conditions better than the original idealized formulations.

In the original treatment subroutine there was no consideration of underflow (the outflow stream containing high solids concentrations due to separation in rotary flow field) in the swirl concentrator, leading to a discrepancy in the mass balance across this unit. This is not realistic and consequently, an underflow rate equal to 1/70th of the inflow rate (obtained from empirical data) has been incorporated for this unit. Also in the treatment section the suspended solids removal efficiency was raised from 22 to 35 per cent to reflect recent reported improvements in performance [5]. This unit will therefore appear more cost effective in the revised model.

The cost estimates in the model for the individual treatment units are based on the general relationship

$$S = AQ^{B}F \quad (4.2)$$

where A is a base cost factor, Q is the design flow rate for the unit (in MGD), B is an indicator of the economy of scale and F is a correction factor for the specific time and locality. Cost data for stormwater facilities in North America are rather limited and a reliable statistically significant

---

* where settling efficiency is given by $\eta = 0.82e^{-Q/2780}$ and Q is the overflow rate in USgpd/ft^2

data base is not yet available. At present much of the activity in storm-water storage and treatment is instigated by governmental agencies and consequently the available figures may not reflect all the costs involved in construction. Cost curves and equations were developed utilizing the publications of Environment Canada, Provincial MOE's and the U.S.-E.P.A. as well as bid construction costs and equipment manufacturers' estimates. The costs incorporated in the revised model, where relevant, include estimates of electrical piping, roads and other related capital expenditures. The treatment plants assessed in this study ranged in capacities from 2.2 to 44.4 cfs. The model can accommodate design flows up to 770 cfs. Obviously there will be a considerable degree of uncertainty in cost projections for the higher flow range, but generally the relative costs of different treatment combinations should remain realistic. The present model employs the ENR Index, published in Engineering News Record as a means of standardizing estimates. This is considered to be the best available index, although the ENR may be somewhat limited in the area of municipal wastewater treatment. The Water Quality Office - Sewage Treatment Plant(WQO-STP) Index has proved successful in adjusting treatment plant costs to a common time period, but at present insufficient data exists to perform a comparison. Table 7.1 summarizes the capital cost functions, included in the revised model for the individual treatment options. These were adjusted for Toronto, March 1975, the ENR Index being 2033.

7.4 TESTING STORAGE/TREATMENT OPTIONS

Each storage and treatment unit was tested individually and in series with other units following the modifications to the subroutine. This involved running the STORAG/TREAT block with a test hydrograph and pollutograph for each process. Some of the untreated and treated pollutographs for the various units are compared in Figures 7-3 to 7-6.

A full scale test of the modified STORAG/TREAT and TRCOST routines was conducted using the storm of September 5, 1971 with data for the Bannatyne combined sewer system. Figure 7-7 illustrates the effect of the storage basin in attenuating the inflow hydrograph (which is output generated by the TRANSPORT block). The treatment plant capacity was specified as 54 cfs. This involves a small overflow over a period of about 10 minutes as can be seen from the flat section of the hydrograph computed at the inflow to the treatment units. There is also some initial removal and dilution of the three pollutants simulated in the storage basin. The increases in the concentration of these pollutants towards the end of the simulation period are explained by the diminishing dilution of dry weather flow, as the magnitude of storm flow declines.

The subsequent performance of a treatment process consisting of swirl concentration, microstraining and chlorine disinfection is illustrated in Figure 7-8 The simulated concentration of BOD in the discharge stream is always less than 10 mg/l compared with peak concentrations in excess of 200 mg/l in the inflow to storage. The reduction in suspended solids is not so dramatic, with a peak concentration of about 120 mg/l in the discharge. However in general, these results appear reasonable. The average coliform concentration in the discharge is reported as 3.4×10^6 MPN/100 ml (not plotted). Thus, even after dilution in the stream, it is possible that the effluent from this combination of treatment processes would not result in primary contact bacteriological standards in the receiving water being satisfied.

A summary of the capital, annual operating and storm event costs estimated in the COST subroutine for the treatment units specified in the example, is shown in Table 7.2.

7.5 CONCLUSIONS

(a) Modifications have been made in the STORAG and TREAT subroutines to better reflect current data on unit performance.

(b) Each treatment unit has been tested in order to ensure the model is fully operational and that variations in the input stream are reflected reasonably in the output stream. The model may be used for the comparison of alternative pollution abatement facilities.

(c) The TRCOST subroutine has been modified in order to better reflect Canadian prices. At present this can only give the planner a general view of the relative costs involved with various treatment alternatives. However, as more statistically useful data becomes available following the installation of new facilities and pilot plants, the treatment and cost estimate models should be updated to provide more accurate indications of both efficiency and cost of treatment.

(d) The time step employed in the treatment computations is usually the same as that employed in the TRANSPORT Block (typically 5 min.). Since the equations in the TREAT subroutine are based mainly on data obtained from units operating at steady state it is considered that the use of a longer time step (say 30 min.) in treatment computations could be employed without a significant loss of accuracy.

TABLE 7.1

CAPITAL COST FUNCTIONS

(ENR = 2033, F = 1.0)

Treatment Options	Cost Functions
1. Bar Racks	$40400\ Q^{.61}F$
2. Swirl Concentrator	$3330\ DIA.F$
3. Inlet Pumping	$58500\ Q^{.84}F$
4. Dissolved Air Flotation	$110000\ [Q(1 + RECIRC)]^{.43}F$, $Q \leq 10$ MGD $35600\ [Q(1 + RECIRC)]^{.92}F$, $Q > 10$ MGD
5. Fine Screens	$1857QF$
6. Sedimentation Tanks	$5840000\ (^{Q}/R)^{.55}F$
7. Microstrainer	$67000\ Q^{.46}F$, $Q \leq 20$ MGD $27275\ Q^{.76}F$, $Q > 20$ MGD
8. High Rate Filters	$56000\ Q^{.80}F$
9. Biological Treatment	$1400000Q^{.69}F$
10. Effluent Screens	$11200\ Q^{.56}F$
11. Outlet Pumping	$58500\ Q^{.84}F$
12. Chlorine Contact Tank	$28100\ Q^{.60}F$
13. High Rate Disinfection	$30700Q^{1.0}F$

where Q = design flow rate (mgd)*

F = correction factor

DIA = unit diameter

R = overflow rate (gals*/ft^2/day)

RECIRC = recirculated flow

*All volumes in U.S. gallons.

Cont'd

TABLE 7.1 (Cont'd)

USEPA - SWMM CAPITAL COST FUNCTIONS

FOR COMPARISON

(ENR = 2033, F = 1)

Treatment Options	USEPA SWMM Cost Function
1. Bar Racks	$22363 + 20330\ Q^{.625}\ F$
	$Q \leq 100$ MGD
	$22363 + 3499\ Q\ F$
	$Q \geq 100$ MGD
2. Swirl Concent.	3140 Dia. F.
3. Inlet Pumping	$38680\ Q^{.58}\ F$
	$Q \leq 20$ MGD
	$24755\ Q^{.73}\ F$
	$20 < Q \leq 100$ MGD
	$7134\ Q\ F$
	$Q > 100$ MGD
4. Dissolved Air Flotation	$45016\ Q\ F$
5. Fine Screens	$18570\ Q\ F$
6. Sedimentation Tanks	$87419\ (700Q/R)^{.91}\ F$
	$Q \leq 100$ MGD
	$874\ Q\ (70000/R)^{.91}\ F$
	$Q > 100$ MGD
7. Microstrainers	$14699\ QF$
8. High Rate Filters	$163332\ Q^{.67}\ F$

Cont'd

TABLE 7.1 (Cont'd)

USEPA - SWMM CAPITAL COST FUNCTIONS

FOR COMPARISON

(ENR = 2033, F = 1)

Treatment Options	USEPA SWMM Cost Function
9. Biological Treatment	79868 Q F
10. Effluent Screen	$13763\ Q^{.625}\ F$ $Q < 100$ MGD 2759 Q F $Q \geq 100$ MGD
11. Outlet Pumping	$37306\ Q^{.625}\ F$
12. Contact Tank	1295 Q F
13. High Rate Disinfection	3070 Q F

TABLE 7.2

SUMMARY OF TREATMENT COSTS

INPUT

ASSUMED FUTURE ENGINEERING NEWS RECORD INDICES
CONSTRUCTION - 20 CITY AVERAGE

YEAR	ENR INDEX
1970	1314
1971	1346
1972	1378
1973	1410
1974	1442
1975	1474
1976	1506
1977	1538
1978	1570
1979	1602
1980	1634

COST PARAMETERS . .

INTEREST RATE	=	7.00 Percent
AMORTIZATION PERIOD	=	25 Years
CAP. RECOVERY FACTOR	=	0.0858
YEAR OF SIMULATION	=	1974
SITE LOCATION FACTOR	=	1.0000

UNIT COSTS . .

LAND	=	1000.00 $/ACRE
POWER	=	0.020 $/KWH
CHLORINE	=	0.200 $/LB
POLYMERS	=	1.200 $/LB
ALUM	=	0.03 $/LB

OUTPUT

TOTAL LAND REQUIREMENT = 0.90 ACRES

TREATMENT	LEVEL	CAPITAL COSTS INSTAL	CAPITAL COSTS LAND	ANNUAL COSTS INSTAL	ANNUAL COSTS LAND	ANNUAL COSTS MIN MAINT	STORM EVENT COSTS CHLORINE	STORM EVENT COSTS CHEM	STORM EVENT COSTS OTHER
BAR RAK & SW.C.	1	213804.	72.	13764.	5.	1604.	0.	0.	28.
NO INLET PUMPING	2	0.	0.	0.	0.	0.	0.	0.	0.
BYPASS LEVEL 3	3	0.	0.	0.	0.	0.	0.	0.	0.
MICROSTRAINERS	4	364931.	774.	31315.	54.	7299.	0.	0.	34.
NO EFFL. SCREENS	5	0.	0.	0.	0.	0.	0.	0.	0.
NO OUTLET PUMPS	6	0.	0.	0.	0.	0.	0.	0.	0.
CONTACT TANK	7	246761.	141.	21175.	10.	4935.	31.	0.	15.
	SUB-TOTAL	$825496.	$988.	$66254.	$69.	$13838.	$ 31.	$ 0.	$ 76.
	TOTAL	$826484.			$80160.			$107.	
	TOTAL PER TRIB ACRE	$1524.			$ 148.			$ 0.	

REFERENCE - CHAPTER 7

1. "Storm Water Management Model" Volume 1 - Final Report, prepared for the U.S. - EPA by Metcalf and Eddy, Inc., July 1971, Water Pollution Control Research Series 11024DOC07/71.

2. R. Sullivan, M. Cohn, J. Ure, F. Parkinson, "The Swirl Concentrator as a Grit Separator Device", Environmental Protection Technology Series, Report EPA-670/2-74-026, June 1974.

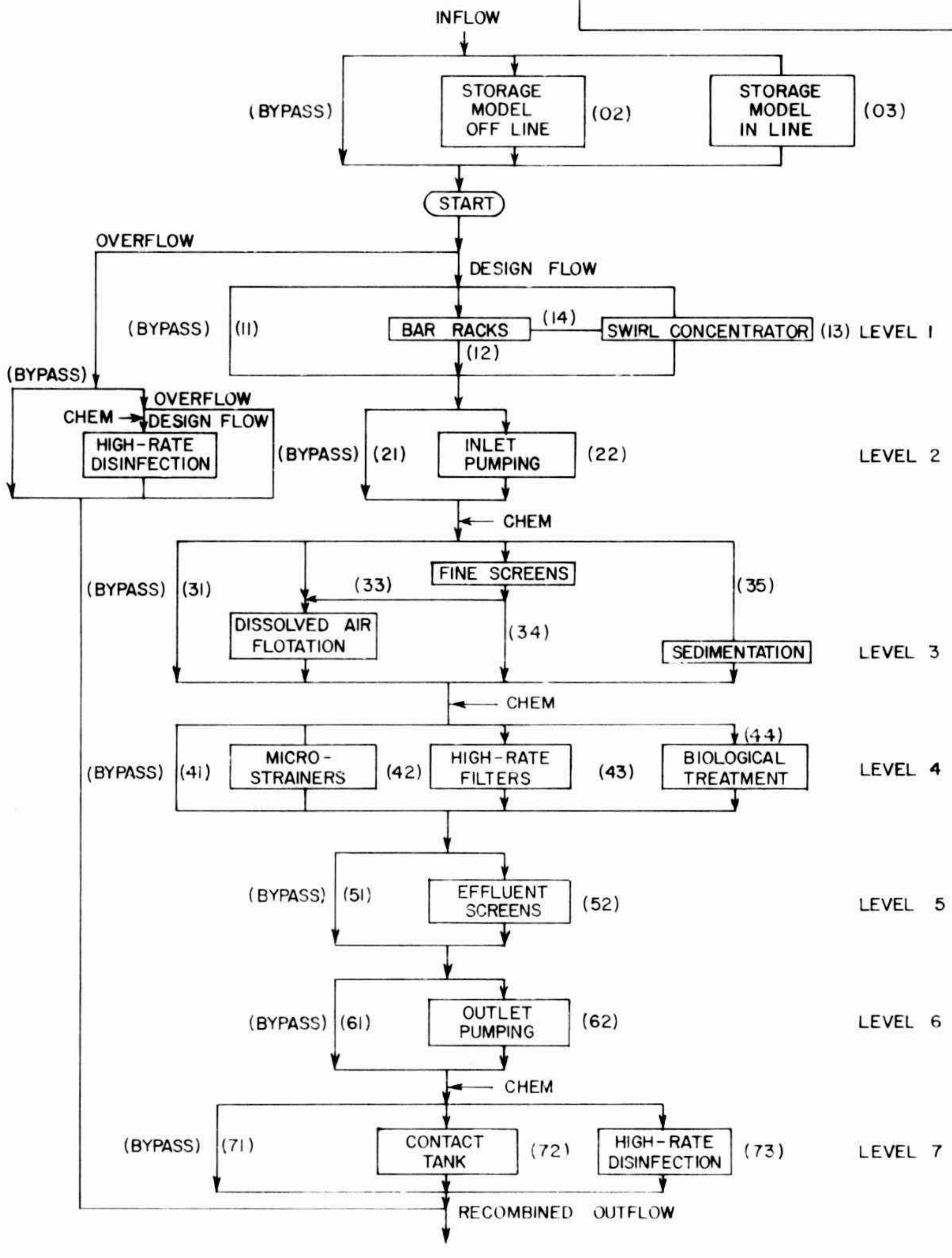
3. "Urban Storm Water Management and Technology, An Assessment" prepared for the U.S. - EPA by Metcalf and Eddy, Inc., December 1974, EPA-670/2-74-040.

4. Smith, R., "Preliminary Design of Simulation of Conventional Waste Water Renovation Systems Using the Digital Computer," U.S. Dept. of the Interior, FWPCA, 1968.

5. Richard H. Sullivan, Project Director, "The Swirl Concentrator as a Combined Sewer Overflow Regulator Facility", Environmental Protection Technology Series, Report EPA-R2-72-008, September 1972.

OPTIONS AVAILABLE IN TREATMENT MODEL

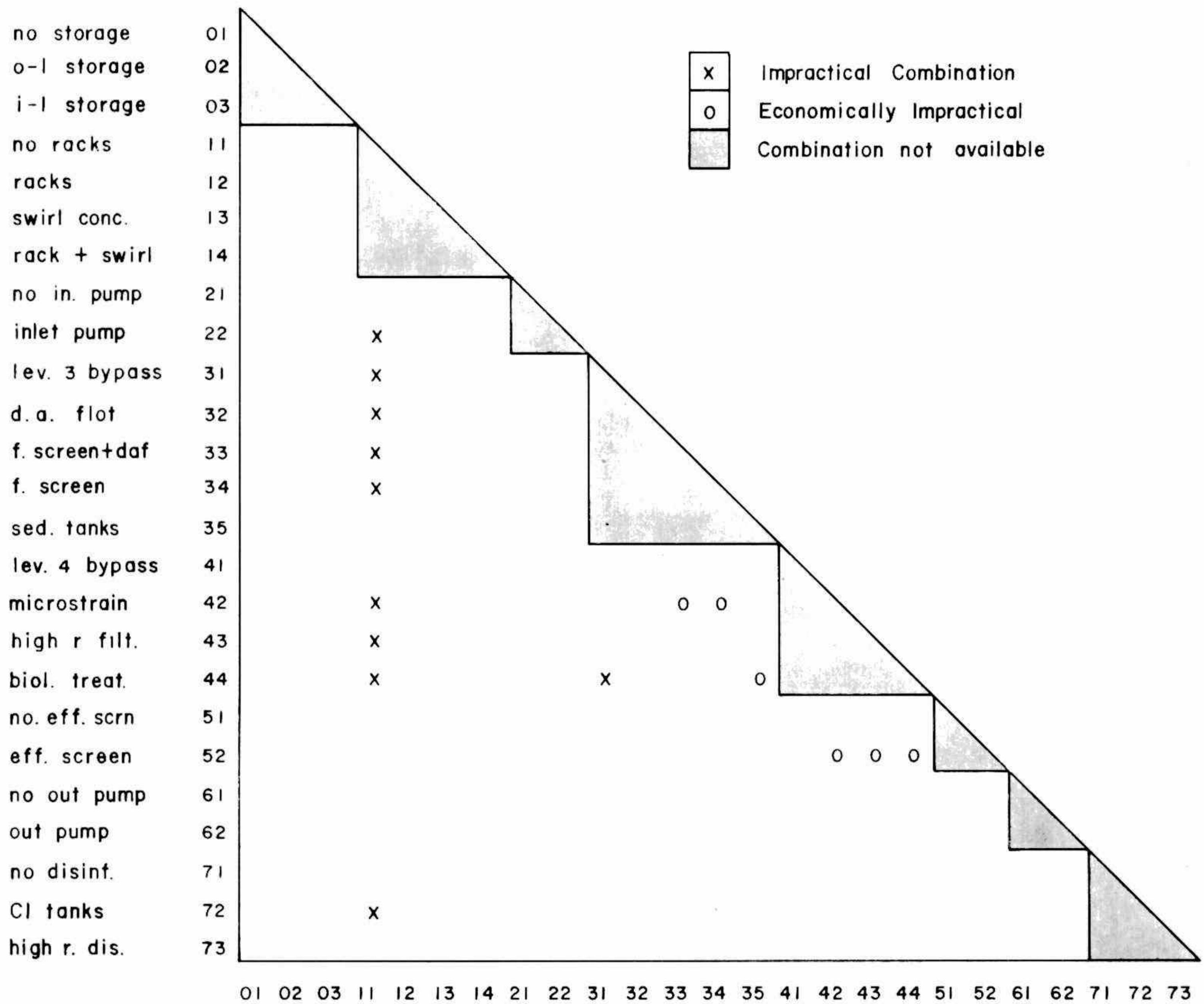
FIG. 7.1



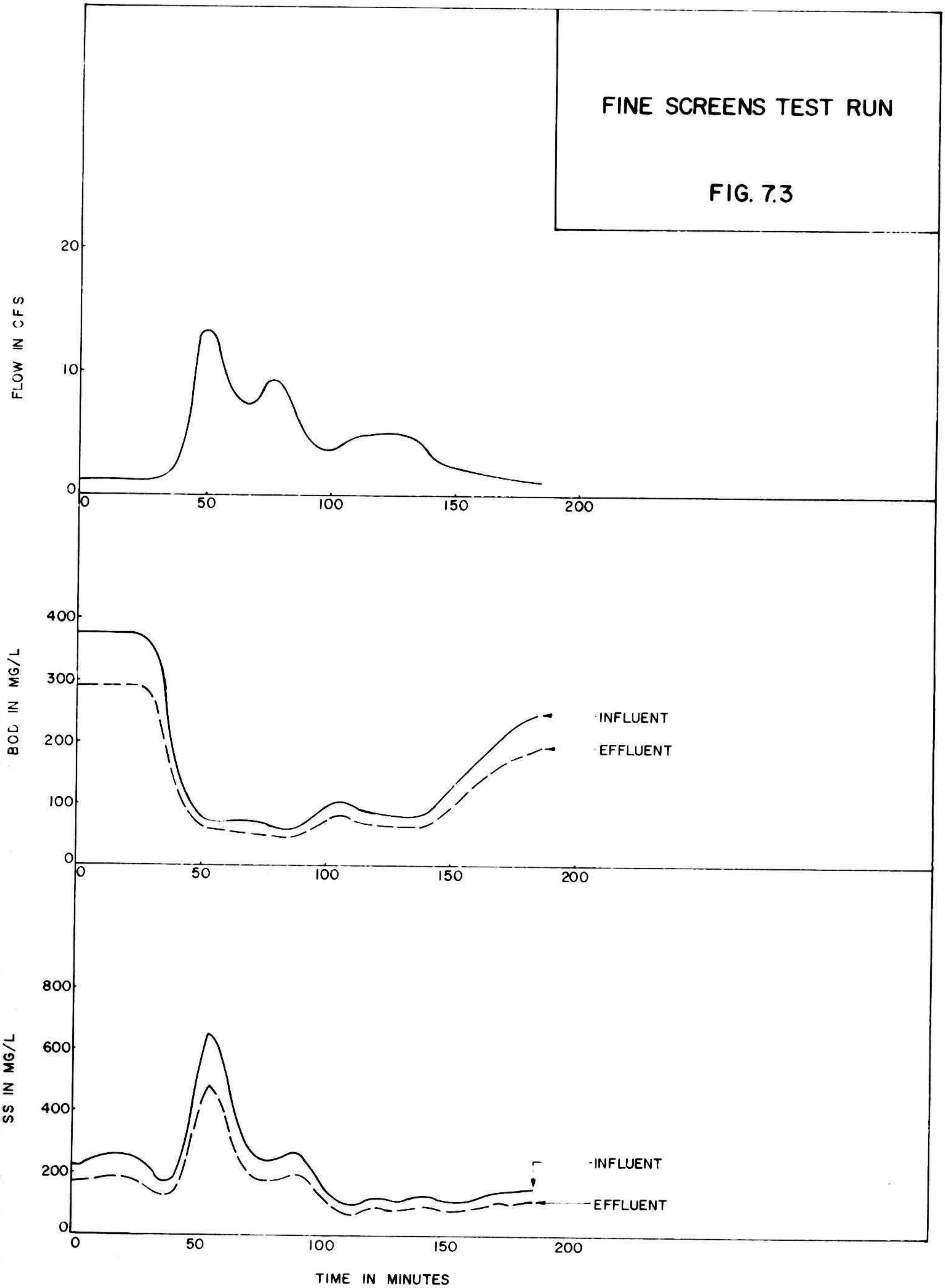
STORAGE/TREATMENT OPTIONS

FEASIBLE COMBINATIONS*

FIG. 7.2

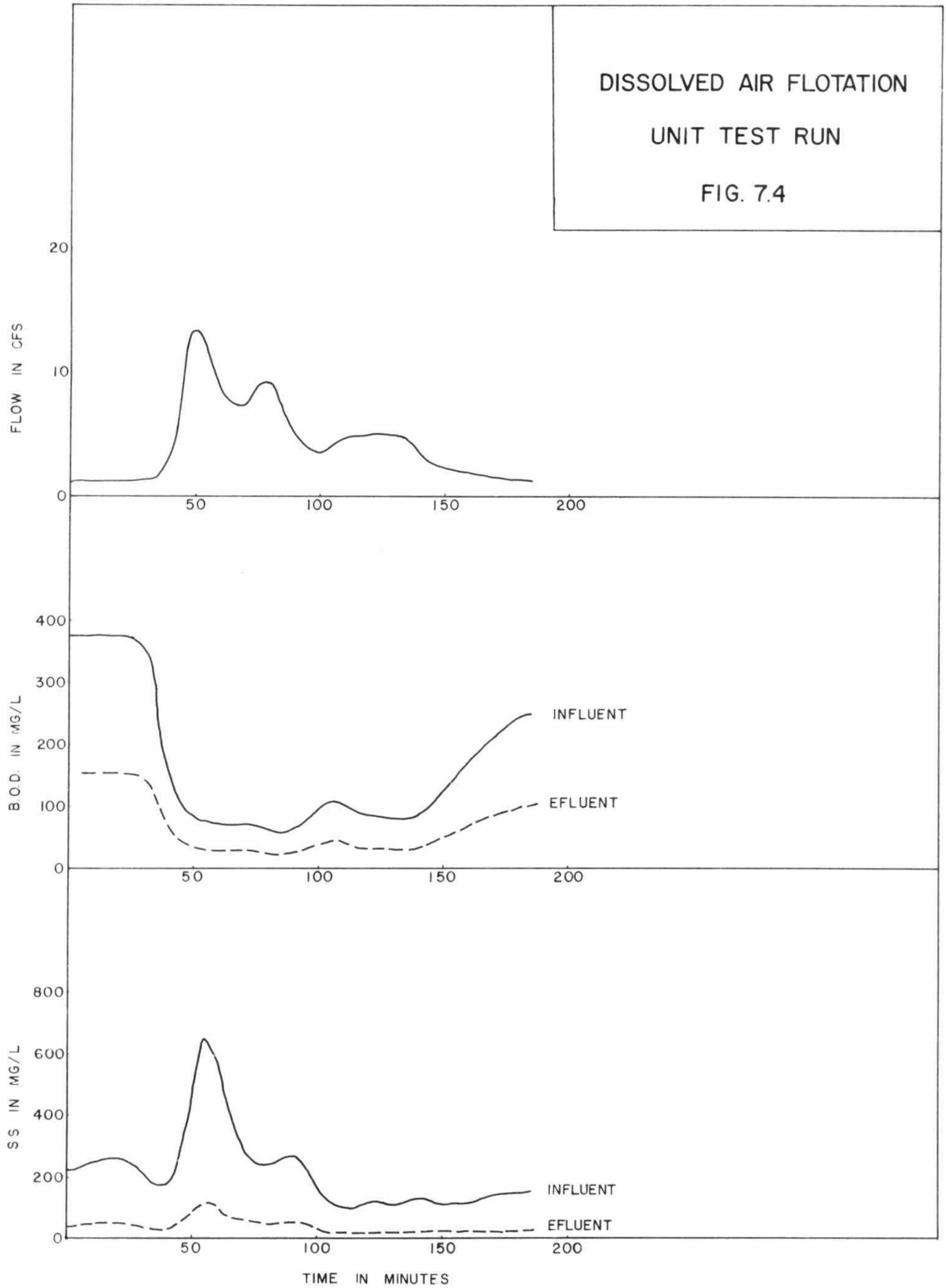


* Modified From Original SWMM Users Manual



FINE SCREENS TEST RUN

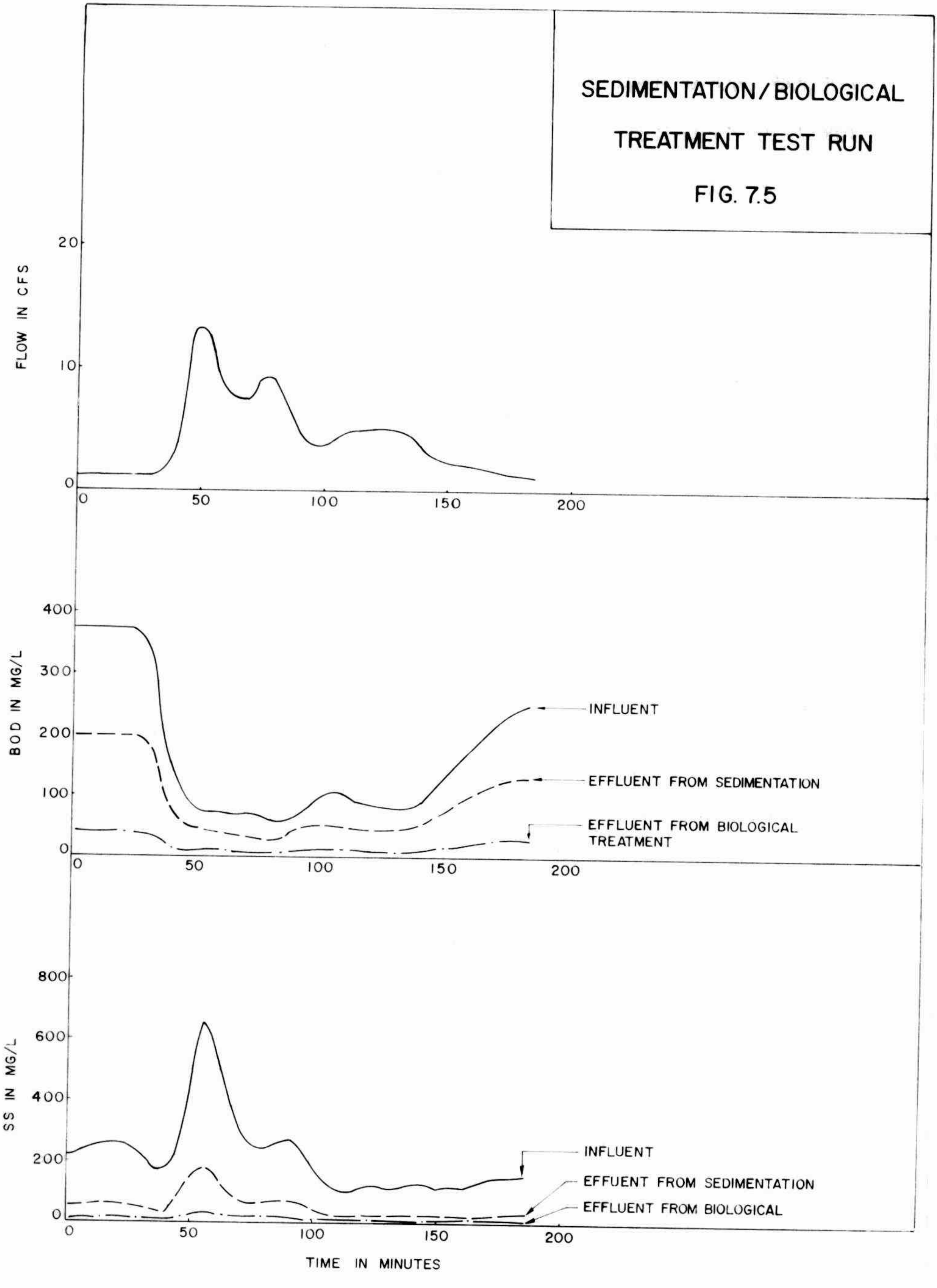
FIG. 7.3



DISSOLVED AIR FLOTATION

UNIT TEST RUN

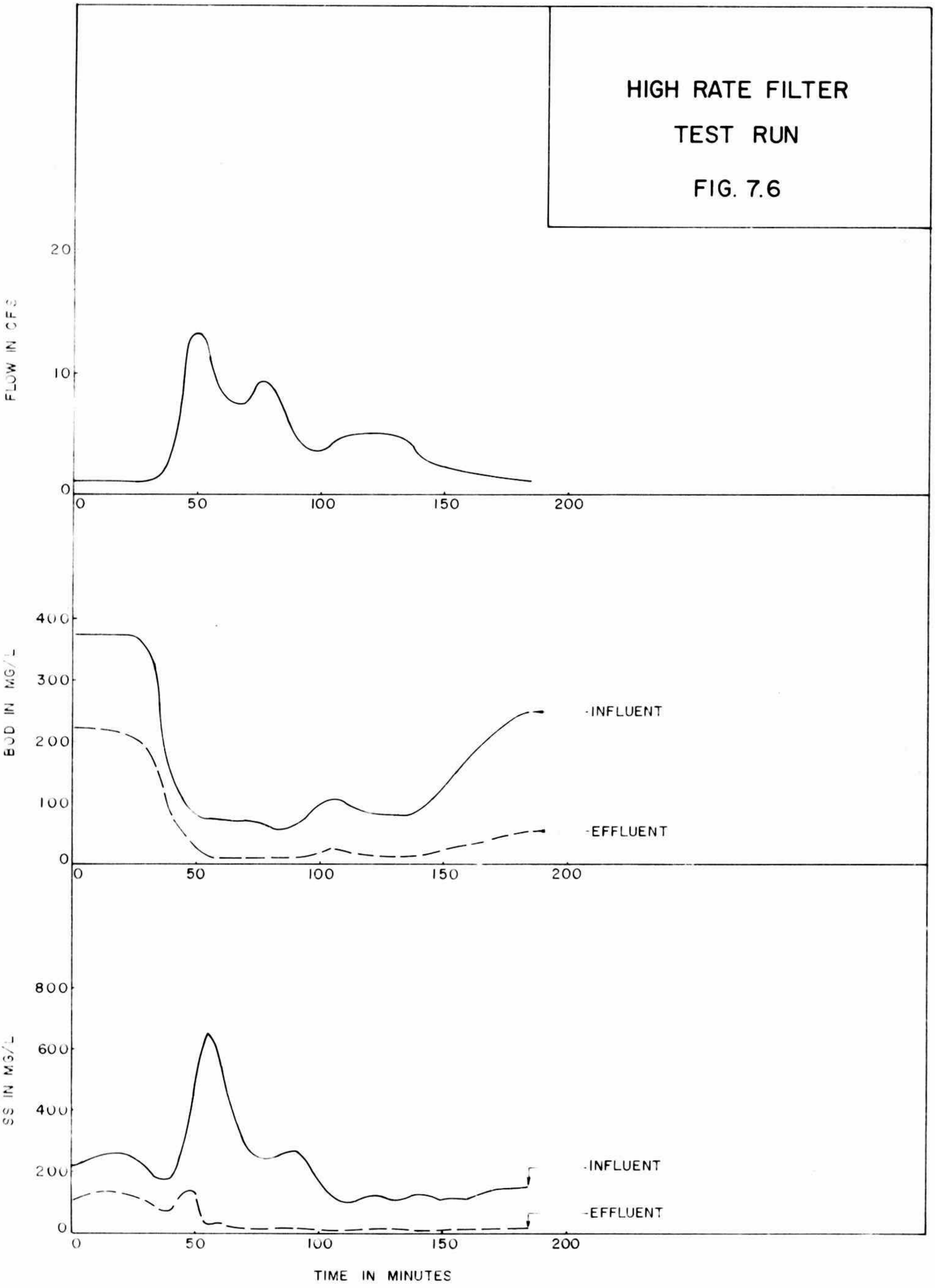
FIG. 7.4

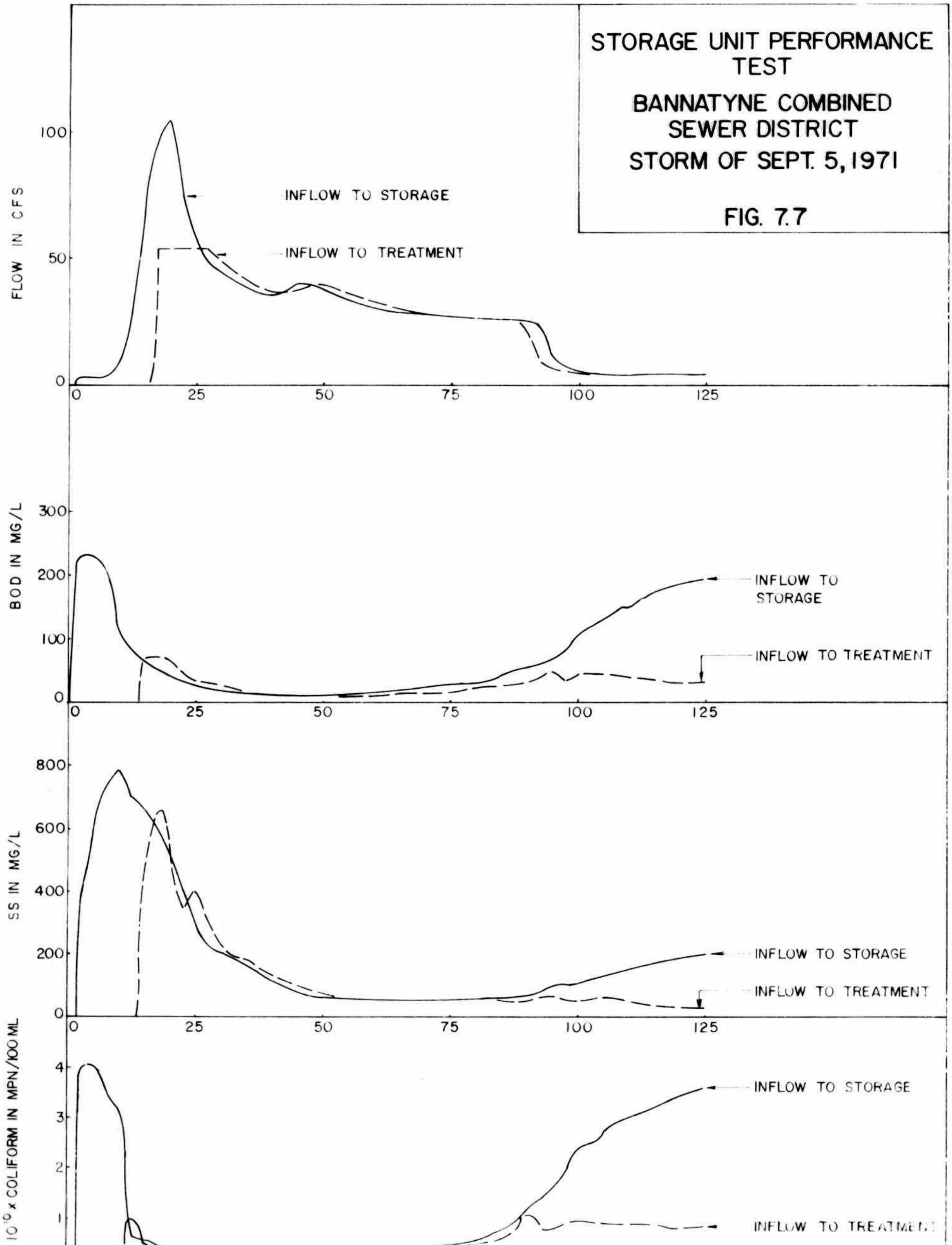


SEDIMENTATION / BIOLOGICAL TREATMENT TEST RUN

FIG. 7.5

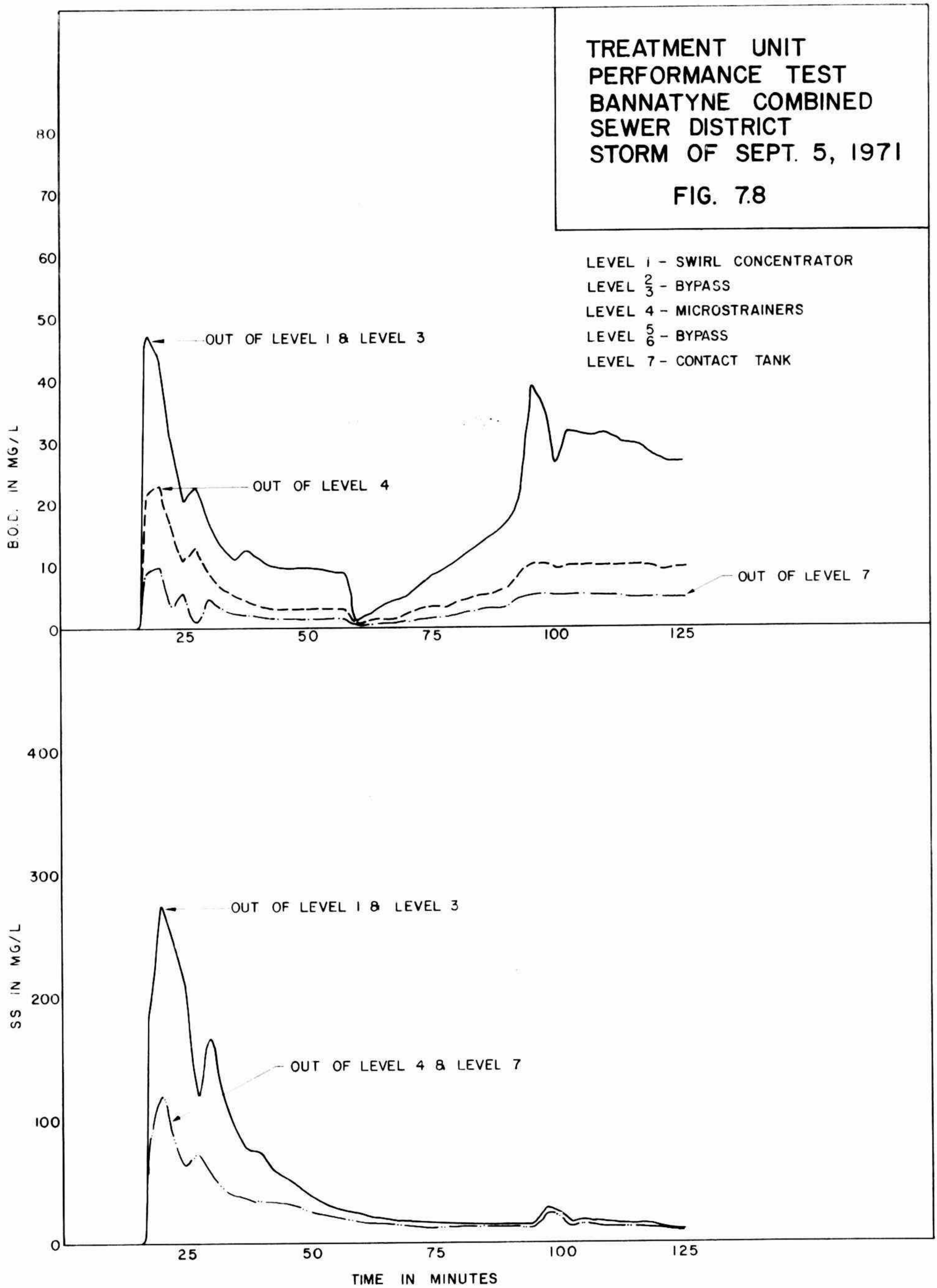
HIGH RATE FILTER
TEST RUN
FIG. 7.6





STORAGE UNIT PERFORMANCE TEST
BANNATYNE COMBINED SEWER DISTRICT
STORM OF SEPT. 5, 1971

FIG. 7.7



TREATMENT UNIT
PERFORMANCE TEST
BANNATYNE COMBINED
SEWER DISTRICT
STORM OF SEPT. 5, 1971
FIG. 7.8
LEVEL 1 - SWIRL CONCENTRATOR
LEVEL 2/3 - BYPASS
LEVEL 4 - MICROSTRAINERS
LEVEL 5/6 - BYPASS
LEVEL 7 - CONTACT TANK
OUT OF LEVEL 1 & LEVEL 3
OUT OF LEVEL 4
OUT OF LEVEL 7
B.O.D. IN MG/L
80
70
60
50
40
30
20
10
0
25
50
75
100
125
400
300
200
100
0
SS IN MG/L
OUT OF LEVEL 1 & LEVEL 3
OUT OF LEVEL 4 & LEVEL 7
TIME IN MINUTES

SNOWMELT QUANTITY AND QUALITY

CHAPTER 8

CHAPTER 8
SNOWMELT QUANTITY AND QUALITY

8.1 GENERAL

Snowmelt can be accompanied by intense rainstorms in spring or occasionally in late fall. This may lead to significant flooding in urban areas, even in the case of small basins. In urban areas of Canada, water quality during winter may be affected by application of chemical de-icing agents. In some cases the snowpack acts to retain various pollutants which are then released to the receiving water body during the melt period. The prediction of the time of release of melt-water in some cases is complicated by the fact that chemical additives and the mechanical removal of snow affect the natural snowmelt processes. The amount and distribution of snow on streets and other impervious and pervious surfaces must be known or estimated prior to an assessment of melt quantities.

The basic problem is therefore to simulate the quantity and quality of runoff water resulting from snowmelt or a combination of rainfall and snowmelt. The complete reviews of the literature concerning the quantity and quality of snowmelt are given in Volume 2. In the following sections the results and conclusions of the literature surveys are discussed and the model developed for integration with the SWMM is described.

8.2 SELECTION OF A SNOWMELT QUANTITY MODEL

8.2.1 *Summary of the Literature Review*

The available snowmelt models can be broadly classified as follows:

(a) Detailed heat budget models requiring extensive meteorological input data.

(b) Regression type models of an "intermediate" nature accounting for the most important meteorological parameters.

(c) Empirical models such as the degree-day method.

The detailed models are based on the following generalized heat budget equations:

$$H_{melt} = H_{snow} - H_{cc}$$

$$H_{snow} = H_{swr} + H_{lwr} + H_{ca} + H_{cond} + H_{g} + H_{p} \qquad (8.1)$$

where H_{swr} = short wave radiation absorbed by the snow pack

H_{lwr} = net longwave radiation exchange with the snow pack

H_{ca} = convective heat transfer from the air

H_{cond} = heat of vaporization released by condensate on the snow surface

H_{g} = heat conducted from the ground

H_{p} = heat content of precipitation

H_{snow} = change in heat storage of the snow pack

H_{melt} = heat available to melt a given amount of snow

H_{cc} = "cold content" of the snow pack

When a snow pack is below the freezing point it has a negative heat content. The heat required to raise the temperature of the snow pack to $0^{o}C$ can be computed from equation (8.1). Any additional heat causes the snow to melt. The most important factors governing the heat transfer processes involved in equation (8.1) are air temperature and net radiation.

Several existing detailed mathematical computer models are based on various forms of equation (8.1). These include Amorocho's model, the Stanford Watershed model and the Hydrocomp simulation program [1,2,3]. (These snow accumulation and melt models are described in Volume 2). The various mathematical relationships employed in most of the heat budget models are based on the work of the U.S. Corps of Engineers [4]. Sophisticated models generally require a considerable number of input parameters, including the following:

- short wave radiation
- air temperature
- dewpoint temperature
- wind velocity
- precipitation
- cloud cover fraction

The computational time interval for such models is usually limited to one hour as the necessary meteorological data are usually not available at more frequent intervals than this.

Several empirical snowmelt models have been developed based on regression relationships. These models incorporate the most important meteorological parameters, but are not generally applicable over a range of different geographical areas.

The method which has received the widest application due to its relative simplicity is the so-called degree-day method. A degree-day is defined as a deviation of one degree from a given datum temperature over a 24 hour period. Snowmelt is given by the following equation:

$$MELT = C\,(T_a - T_{sn}) \tag{8.2}$$

Where C = Melt coefficient for the particular area,
T_a = air temperature,
T_{sn} = base temperature for melt,
MELT = snowmelt in inches (or centimeters) per day

Equation (8.2) may be adapted for the computation of melt during time periods of less than one day.

The degree-day method gives good results in view of the simplifying assumptions and data limitations. For example, Anderson compared a detailed heat budget method with the degree-day method and concluded that both methods gave quite similar results. Anderson has developed a model which combines the advantages of a degree-day computation with the flexibility of the heat budget method. This model is described in more detail in Section 8.2.2. [5,6].

One of the major obstacles to an accurate simulation of the snowmelt runoff process is the need to satisfactorily account for the movement of meltwater through the pack and for the resulting runoff. This problem is sometimes aggravated by unusual infiltration rates due to partially frozen topsoil. The literature survey indicated that at present there are insufficient data to develop a runoff model sophisticated enough to account for all these phenomena. Consequently the SWMM runoff equations were modified for use in the present model.

8.2.2 *Selection of a Snowmelt Quantity Model*

The selection of an appropriate snowmelt model for integration with the SWMM was based on the following considerations:

(a) A time step of 1 hour is sufficiently accurate for the computation of the total volume of meltwater during periods of no rainfall. (Intermediate values required for short time steps in SWMM calculations are interpolated). This accuracy is also sufficient for hydrograph computations during rainfall periods since the rainfall intensity for the short time intervals involved is normally significantly greater than the snowmelt intensity.

In general, for single event modelling, the snow pack distribution and physical parameters are known or may be determined in order to define initial conditions prior to a storm. The problem is then to describe the physical changes in the snow covered areas during the rainstorm and the resulting effects on the runoff.

(b) The following assumptions can also be made for single-event simulation:

(i) Frozen ground remains frozen during the given event.

(ii) The areal distribution of snow cover can be assumed available as an input. The areal distribution can be accounted for on a sub-catchment basis and this distribution is assumed constant throughout the event simulation. Snow cover in the different subcatchments will be depleted with time during the simulation according to the amounts available at the beginning of the event.

(iii) It is assumed that during the simulation, the melt volume resulting from the application of de-icing salts is negligible. It is assumed that no additional salt is applied during the melt period. The effect of salt on the free water in the snowpack at the beginning of the storm is accounted for in the model input parameters, which can be measured or calibrated for particular event.

(iv) Incoming solar radiation during a rain on snow event is negligible and the incoming longwave radiation equals the blackbody radiation at the ambient air temperature.

(v) The dew point temperature and the temperature of the rain water are equal to the ambient air temperature. The temperature of the melting snow surface is 32°F.

(vi) For single-event simulation of shallow (non-mountain type) snowpacks, the negative heat component of equation (8.1) is assumed to be negligible. That is, heat applied to the snowpack will immediately result in some melt which would be either stored in the snow or become available for runoff.

(vii) The air temperature record and average wind speed are available.

(c) Paved areas are usually cleared of snow in urban areas. Consequently, it is anticipated that, the runoff from these areas, in a subsequent rainfall event, would be greater than from the snow covered areas. The ability to describe bare areas as well as snow covered areas is discussed in the user documentation.

The Anderson snowmelt model was selected on the basis of the above assumptions. During rainfall periods melt is given by the following equation:

$$MELT = (T_a - 32) * (0.007\, P_x \;\Delta T + 7.5\, \gamma\, F\ (u) + 0.00117 \Delta T) + 8.5\, F\ (u) * (e_a - 0.18) \tag{8.3}$$

Where T_a = air temperature (^oF)

$F(u)$ = wind function = $.006\ V \Delta T$

e_a = vapour pressure of air (inches Hg)

P_x = rainfall in inches per hour

γ = .000359 PA Where PA is the air pressure in inches of Hg.

ΔT = time interval in hours

V = wind speed in mph

The general algorithm for computing snowmelt using the above equations is given in Figure 8-1. Air pressure and vapour pressure are calculated by the model using the relationships summarized in Figure 8-1. (Further discussion is contained in Volume 2 and [6]). The melt equation is linear with respect to time. Therefore the melt and associated variables may be determined by interpolation at each of the computational time steps used in the RUNOFF Block.

Equation (8.3) should be used for rain on snow events or during cloudy periods preceding or following rainfall events. For periods of no rainfall, the degree-day melt equation (8.2) is used to time compute snowmelt by adjusting the melt coefficient to be consistent with the time interval for computation. A base temperature between 25 to 32°F can be selected

depending on the geographical location and prevailing meteorological conditions. A base temperature of 32°F and a degree-hour melt coefficient of 0.003 are assumed as default values. Methods for estimating the base temperature and melt coefficient can be found in the literature (e.g., [7]). It should be noted that use of the degree-day option for rain-free periods will give time averaged snowmelt rates.

8.3 SNOWMELT QUALITY MODEL

8.3.1 *Summary of the Literature Review*

The pollution of snow and melt waters has not been investigated in detail until recently, and therefore, the literature is rather limited. The main efforts to date have been to measure the actual amount of various pollutants in snow in urban areas. The literature found describing the modelling of pollutants in snow or in snowmelt waters is at present very limited.

The major sources of pollutants in snow are related to de-icing chemicals and automobile exhausts. The amount of pollution is related to the degree of urbanization, in Table 8.1. Areas of medium population density (1000-5000 per sq. mile) receive the greatest amount of salting [8].

TABLE 8.1

SALTING RATES IN ONTARIO

Population Density (per sq. mile)	Rates of Salt Application (lbs. per app. per lane-mile)
Less than 1000	75 - 800
From 1000 to 5000	350 - 1800
More than 5000	400 - 1200

Pollutant concentrations in snow are found to be highest along the most heavily travelled roadways.

Chloride concentrations measured in snow from streets in Canadian urban centres have been reported to range from 7 to over 15,000 mg/l. Recent sampling of runoff during the winter season of 1974-1975 at the Brucewood catchment in North York has indicated chloride concentrations as high as 10,300 mg/l. [9,10].

The concentration of lead in snow samples varied up to 113 mg/l along freeways in Ottawa. The lead concentrations in Ottawa storm sewer runoff were sometimes as high as 1 mg/l. Recent samples of runoff in the Brucewood catchment gave measured lead concentrations up to 2 mg/l [11,10].

Generally speaking, chlorides and lead are the most serious pollutants associated with snow and snowmelt waters. Table 8.2 summarizes the ranges of various pollutant concentrations which have been measured:

TABLE 8.2

REPORTED SNOW AND SNOWMELT WATER POLLUTANT CONCENTRATIONS

Pollutant	Observed Range (mg/l)
Chloride	7-15580
Lead	0.02-113
BOD_5	1.4-84
Suspended Solids	1-4020
Organic Nitrogen	0.09-4.3
Nitrate-N	0.01-1.5
Ammonia-N	0.1-0.3
Total Phosphate	0.036-3.6
Sodium	3.6-9480
Phenol	.002-.125

Detailed records of the amount of various pollutants entering the sewer system as a result of snowmelt do not exist. Very few snowmelt quality models have been documented. (Comprehensive reference material is reported in Volume 2)

8.3.2 *Development of Snowmelt Quality Model*

Since specific information concerning the modelling of snowmelt quality is not available, it was decided that the methodology used in the SWMM for storm runoff quality computations should be adopted for snowmelt quality modelling. Chlorides and lead are modelled in addition to other pollutants normally simulated by the SWMM. The basic equation describing pollutant washoff is:

$$\Delta P = P_o \left(1 - e^{-br \cdot \Delta t}\right) \tag{8.4}$$

where P_o = initial surface pollutant load
b = a constant (usually = 4.6)
Δt = time interval
r = runoff rate
ΔP = mass of a pollutant removed in Δt

(These are discussed more fully in Chapter 5).

The initial amount of any pollutant (except chloride) on the ground at the start of a storm can be computed from:

$$P_O = P \cdot G_L \cdot N_D \tag{8.5}$$

where P_o = amount of pollutant at the start of storm
N_D = number of dry days since the last storm
G_L = total length of gutters in hundreds of feet
P = accumulation rate of pollutant lbs./100 ft. of gutter/day

If the number of dry days exceeds the cleaning frequency the surface pollutant load is modified according to the number of sweepings and the efficiency of sweeping. (The equations are given in Chapter 5).

Since the deposition of chlorides is primarily a function of local street salting practices, a different approach is used for the estimation of the surface accumulation of chlorides. In developing the Snowmelt Quality Model it was decided to supply as input to the model the total amount of salt applied to the catchment prior to the event. These data are usually available from municipal records of street salting events. Since different salt application rates are applied in different streets, the amount of salt applied to each subcatchment should be input. Where measurements are not available, the application rates given in Table 8.1 can be used to obtain initial estimates of salt applications.

The total amount of salt applied to the catchment is then converted into chlorides by using the following relationship:

$$P_{o_{cl}} = P_{o_s} \times 600 \times 453.6 \qquad (8.6)$$

where $P_{o_{cl}}$ = total amount of chlorides in mg at the beginning of storm

P_{o_s} = total amount of salt applied prior to the storm in lbs.

and 600 x 453.6 = a unit conversion factor

The amount of chlorides washed off during the storm is then computed in the same manner as for other pollutants.

A description of the required input data formats is given in the User's Manual, Volume 3.

8.4 INTEGRATION AND VERIFICATION OF THE SNOWMELT MODELS

8.4.1 *Integration*

(a) The Snowmelt Quantity Model discussed in Section 8.2.2 (see Figure 8.1) has been integrated with the SWMM RUNOFF

Block. Two equations for snowmelt have been included as options in the integrated model. During rainfall periods melt is given by equation (8.3). Snowmelt in rain-free periods can be computed from the "degree-hour" method, which is simply the degree-day method modified for use with shorter intervals, or by using equation (8.3).

The principal input parameters are:

- wind speed (mph)
- ambient temperature (°F)
- distribution of snow
- snow amount (in inches of water equivalent)

The SWMM RUNOFF block has been modified to account for rainfall plus snowmelt runoff from completely or partially snow covered pervious and impervious areas. The time log to runoff which is affected by the ability of the snowpack to retain a certain amount of free liquid water is approximated by specifying the free water holding capacity of the snowpack as described in the user documentation. When the free water holding capacity has been satisfied by a combination of rainfall plus snowmelt water, runoff is calculated using the existing SWMM RUNOFF methodology. The affect of the snowpack on the runoff can also be taken into account by considering the Manning's 'n' surface roughness coefficient as a calibration parameter. As indicated previously, it should be noted that additional research is required in order to more exactly model the snowmelt plus rainfall runoff phenomena.

A detailed description of input data and formats is given in the User's Manual, Volume 3.

(b) The runoff quality routines used in SWMM have been modified to describe the accumulation and washoff of lead and chlorides associated with the snowpack. The Snowmelt Quality routines have been integrated with the Snowmelt Quantity model and now form a User option in the SWMM RUNOFF Block.

8.4.2 *Testing and Verification of the Anderson Snowmelt Quantity Model*

8.4.2.1 *Testing With Lysimeter Data*

The Snowmelt model was initially tested using data from the Mer Bleue research site in Ottawa. The hourly data available at this site for the winter season of 1973-1974 includes snowpack, melt measurements from a lysimeter with a surface area of about 13 m^2, precipitation and air temperature.

The most critical melt period which occurred during the 1973-1974 period was the spring snowmelt of March 4 to 7 inclusive, when 76% of the accumulated

For the purposes of the simulation, it was assumed that equation (8.3) could be applied $\pm$ 3 hours before and after any rainfall occurrence. This equation is used since the underlying assumptions are the same when the time period between rainfall events is relatively short. For other melt periods the degree-hour equation was used with a melt coefficient of .005 and a base temperature of 33°F. Equation (8.3) should be used for rain on snow events where there may be rainfall preceded or followed by a cloudy period. Otherwise, equation (8.2) would be used).

The time history of this event is shown on Figure 8-2. A total of 3.94 inches of melt (5.54 inches of melt plus rainfall) was calculated by the model for the period March 4 to March 7. For those hours in which rainfall occurred, the total rainfall was over 6 times greater than the calculated amount of melt, even though the rainfall intensities are relatively small.

8.4.2.2 *Verification of the SWMM Snowmelt Quantity Model for the Brucewood Area*

Some winter runoff sampling was undertaken as part of the Brucewood data collection programme [10]. The data from this area in North York Toronto was collected during the unusually mild winter of 1974-75. A limited amount of information describing snow distribution, rainfall and snowmelt is available. Only three events are suitable for modelling purposes:

1) March 12, 1975
2) March 19, 1975
3) February 17, 1975

The snow distribution at the start of each event was estimated from recorded observations. In all three cases the accumulations of snow on the ground were relatively small. Meteorological data were obtained from the Downsview Meteorological Station (about 10 miles away). Unless otherwise stated the SWMM default values were used with Anderson's equation in the simulations discussed below. Only the storm of March 12 is plotted as this has the highest proportion of snowmelt in the runoff.

(a) March 12, 1975 (Figure 8-6) - the recorded and simulated hydrographs agree fairly well. However the peak flows are only about 1 cfs. The average ambient temperature was $34^{o}F$ and a snow water equivalent of 0.43 inches was assumed. 25 percent of the impervious area and 85 percent of the pervious area were covered by snow. The proportion of snowmelt contributing to the runoff was computed as about two thirds.

(b) March 19, 1975 - This storm was of a very low intensity (maximum, 0.12 ins/hr) and the flow rate was less than 0.5 cfs for most of the duration of the event. Only 10 percent of the impervious and 25 percent of the pervious areas were snow covered. The water equivalent depth was taken as 0.1 inches for the simulation. The ratio of computed snowmelt to total runoff recorded was about 1:2. Both the computed and recorded hydrographs oscillated between 0.2 and 0.5 cfs in response to the occasional rainfall increments.

(c) February 17, 1975 - This storm was of fairly low intensity and the proportion of snowmelt in the runoff was very small. The model simulated the peak flows of about 0.5 cfs fairly well; again oscillations in the computed and recorded flows were evident. 20 percent of the impervious and 10 percent of the pervious areas were snow covered and the water equivalent depth of the snow pack was taken as 0.2 inches.

In general the model appears to simulate the peak flows fairly well, even when these are very low, but further testing with more significant events is required for complete validation.

The snowmelt events monitored at Brucewood were all accompanied by rainstorms of relatively low intensity. In order to test the model for higher intensity rain on snow events, the highest intensity part of the rainstorm recorded on August 2-3, 1975 over Brucewood was used as input to the model. A snowpack of 2.5 inches water equivalent was assumed to cover the entire catchment. A temperature of 50^{o}F and a wind speed of 10 mph were also assumed. Typical results obtained from a set of simulations for various catchment conditions are presented on Figure 8-3. Plot 1 represents the hydrograph generated considering only the rainfall. Plot 2 shows the rainfall plus snowmelt hydrograph and plot 3 the hydrograph resulting from snowmelt only. The simulation results indicate that the model is working as expected and also demonstrate that even for higher intensity rainstorms the snowmelt component can account for a significant portion of the hydrograph.

8.4.2.3 *Verification of the SWMM Snowmelt Quantity Model for Toronto International Airport*

In a 1975 study of environmental problems at Toronto International Airport [12] a few snowmelt events were recorded. Some of these records were used and interpreted in this study. The events of March 15 and 16, 1975 were considered suitable for modelling purposes. A detailed discretization (31 subcatchments) of the 400 acre tributary area was used. The area which was mainly comprised of runways and aprons, is about 90 percent impervious. Snow on these areas is ploughed. However it was assumed that the melt from the windrows could be modelled, since this would flow over the adjacent impervious area and into the sewer systems. The degree-hour melt equation was used with a melt coefficient of 0.0043 and a base temperature of 28^{o}F. (These resulted from several calibration runs.) With the exception of the surface roughness coefficient in Manning's equation, which was adjusted to 0.026, the other characteristics of the area were assigned the SWMM default values.

The simulated and measured hydrographs for each of the snowmelt events are shown on Figures 8-4 and 8-5. The agreement is reasonable considering the simplifying assumptions involved.

8.4.3 *Verification of the SWMM Snowmelt Quality Model*

The data used for the verification of the quality model were collected at the Brucewood site during the winter of 1974-75. The information describing these events included in Section 8.4.2.2 will not be repeated here. The results of the simulations are summarized below.

(a) March 12, 1975 (Figure 8-6) - The simulated pollutographs for suspended solids and BOD are of the same general shape and order of magnitude as the recorded curves. The peak suspended solids concentration is underestimated, while the computed BOD is consistently slightly high. The initial chloride concentrations recorded were considerably higher than those computed, but subsequent values agree fairly well. The high initial concentrations are probably a catchbasin effect, since following road salting there is likely to be some immediate snow and ice melt causing an increase in the Cl^- ion concentration in the catchbasins. The salt application factor used for this simulation was 0.75 tons per mile, which was consistent with the available salting records for the area. It should be noted that the initial surface chloride load may be less than that corresponding to the actual application rate, since some chlorides may be dissolved in the immediate melt from the roads.

It was possible to reproduce the recorded lead concentrations reasonably well, with the exception of the peak values (Figure 8-7). However this involved specifying a very high

lead accumulation rate (0.38 lbs/100 ft/day) to the model. This is probably not a reasonable estimate when compared with other pollutant accumulation rates. The apparent discrepancy may possibly be due to the localized accumulation of lead in the immediate curb area. The actual contributing area for lead compounds may be fairly small and may contribute only during the first part of an event. Runoff from more remote areas of the subcatchment may not contain significant quantities of lead. Consequently the total amounts of lead on the catchment compared with those of other pollutants may be fairly small, but still contribute significantly to runoff contamination. The model may be used to compute peak concentrations of lead, but should not usually be applied for determination of total loads of this constituent. It is possible that additional washoff equations may have to be incorporated in the model when a greater data base is available.

For the remaining simulations (March 19, 1975 and February 17, 1975) suspended solids were reproduced fairly well. BOD was overestimated in both cases and the recorded values may have been supressed by the high chloride concentrations measured in the runoff. Chloride levels were simulated fairly well with calibration being limited to small variations in the salt application factor.

8.5 CONCLUSIONS

(a) A Snowmelt Quantity model has been selected and integrated with the SWMM RUNOFF block. Preliminary applications based on a limited amount of data indicate that the model can adequately simulate snowmelt and rain-on-snow events.

(b) A Snowmelt Quality model, based on the SWMM pollutant washoff equations, has been integrated with the SWMM RUNOFF block. There are insufficient data for comparisons between model simulations and measurements and specific conclusions cannot yet be established.

(c) More synoptic sets of data describing snow distribution, snowpack quality, rainfall, snowmelt and runoff quality should be collected. Further verification of the SWMM snowmelt routines should be carried out.

REFERENCE - CHAPTER 8

1. Amorocho, J., and Espildora, B, "Mathematical Simulations of the Snow Melting Processes", University of California, Davis, No. 3001, February 1966.

2. Crawford, N.H., and Linsley, R.K., "Digital Simulation in Hydrology, Stanford Watershed Model IV", Dept. of Civil Engineering, Stanford University, July 1966, Tech. Report No. 39.

3. Hydrocomp Simulation Programming Operations Manual, 2nd Edition, Palo Alto, California, February 1972.

4. U.S.C.E. "Snow Hydrology", North Pacific Division, U.S. Army Corps of Engineers, Portland, Ore., 1956.

5. Anderson, E.A., "Development and Testing of Snowpack Energy Balance Equations", WRR, Vol. 4, No.1, February 1968.

6. Anderson, E.A., "National Weather Service River Forecast System Snow Accumulation and Ablation Model", NOAA Technical Memorandum NWS Hydro-17, U.S. Dept. of Commerce, 1973.

7. Gray, D.M., (ed.), "Principles of Hydrology", Canadian National Committee for the International Hydrological Decade, 1970.

8. Unpublished Report "Municipal Snow Quality Study 1973-74", Ontario Ministry of the Environment.

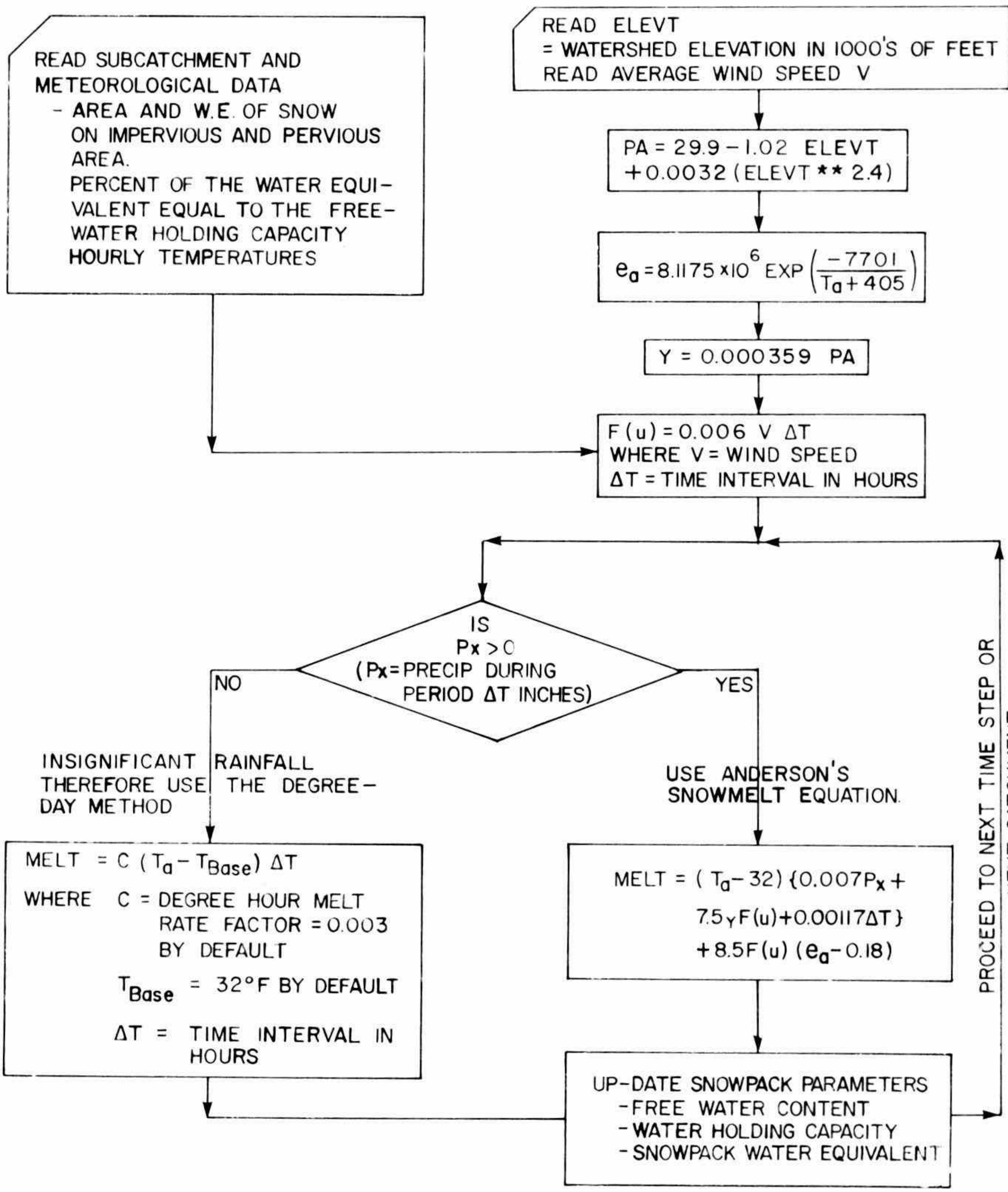
9. "Results of Municipal Snow Quality Study - Winter 1972-73", Unpublished Report by The Ontario Ministry of the Environment, June 1973.

10. Unpublished Draft "Report on the Brucewood Monitoring Programme January 1, 1975 - May 15, 1975", - Prepared by James F. MacLaren Ltd. for Canada Centre for Inland Waters, Burlington, Ontario.

11. LaBarre, N., Milne, J.B., and Oliver, B.G., "Land Contamination of Snow", Water Research, 1, 1973.

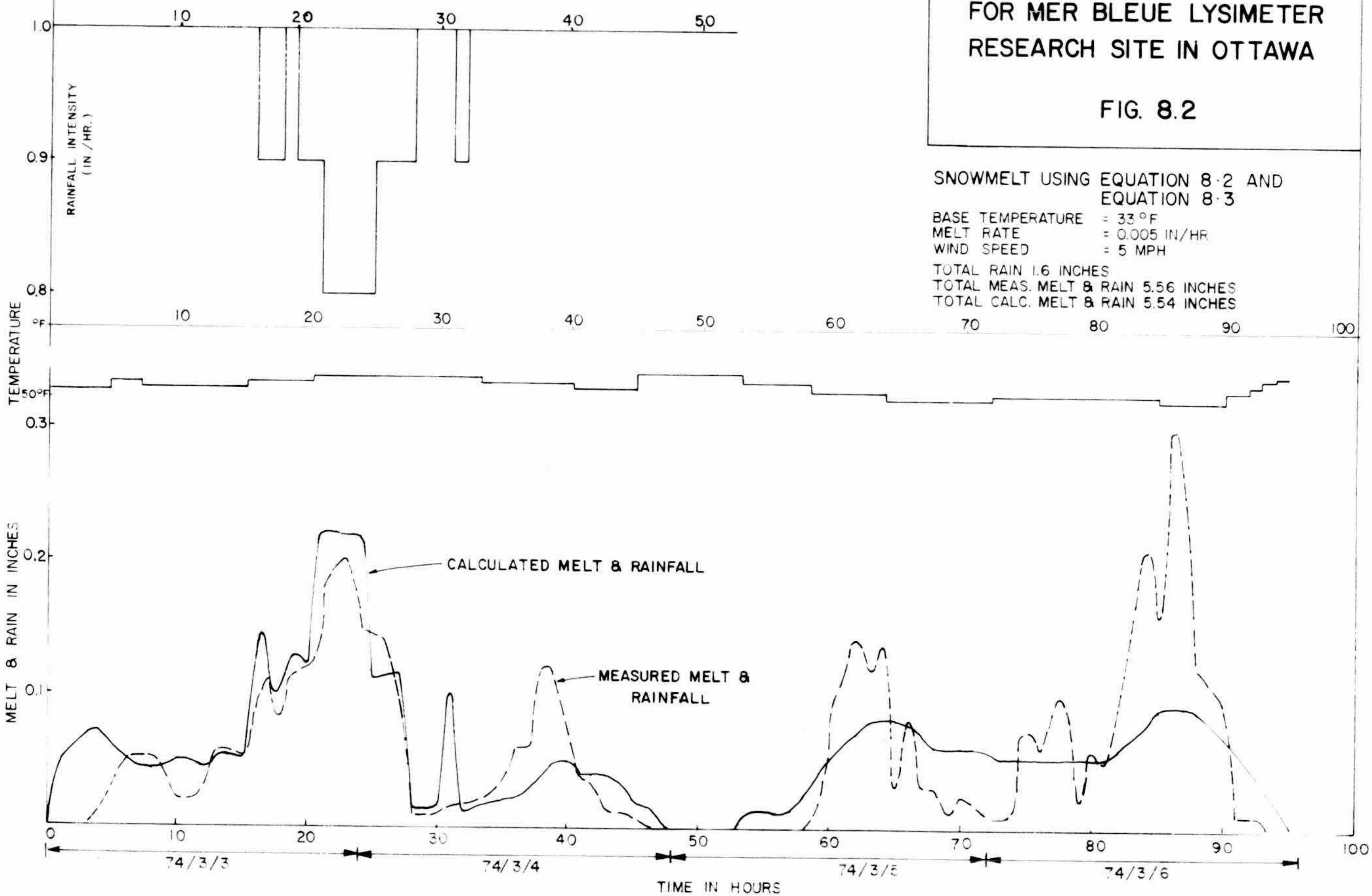
12. "A Study of Environmental Problems at Toronto International Airport", A Report Prepared by James F. MacLaren Ltd. for Department of the Environment, July 1975.

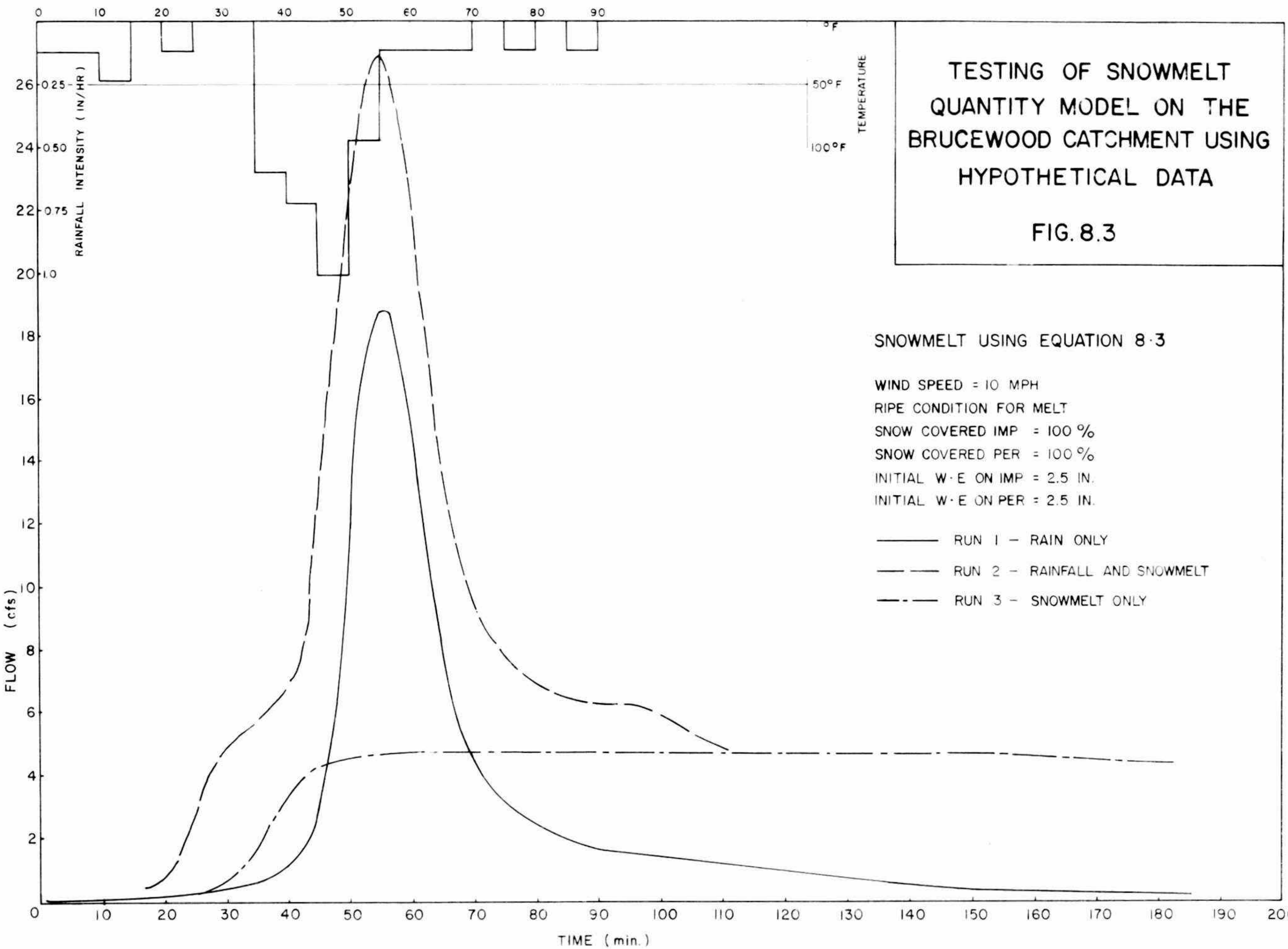
GENERAL ALGORITHM
FOR SNOWMELT
QUANTITY SUBROUTINE
FIG. 8.1



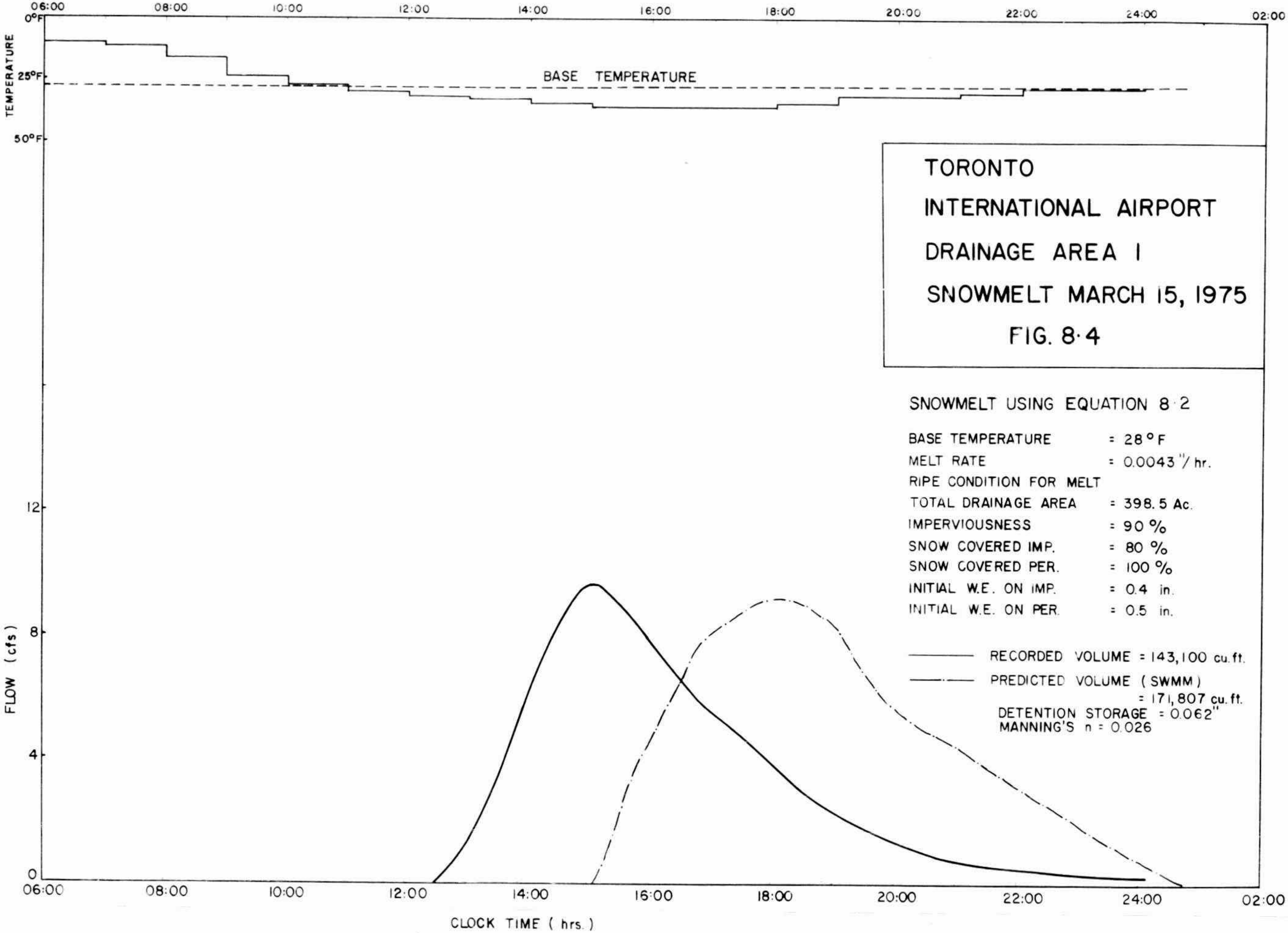
CALCULATED & MEASURED MELT FOR MER BLEUE LYSIMETER RESEARCH SITE IN OTTAWA

FIG. 8.2

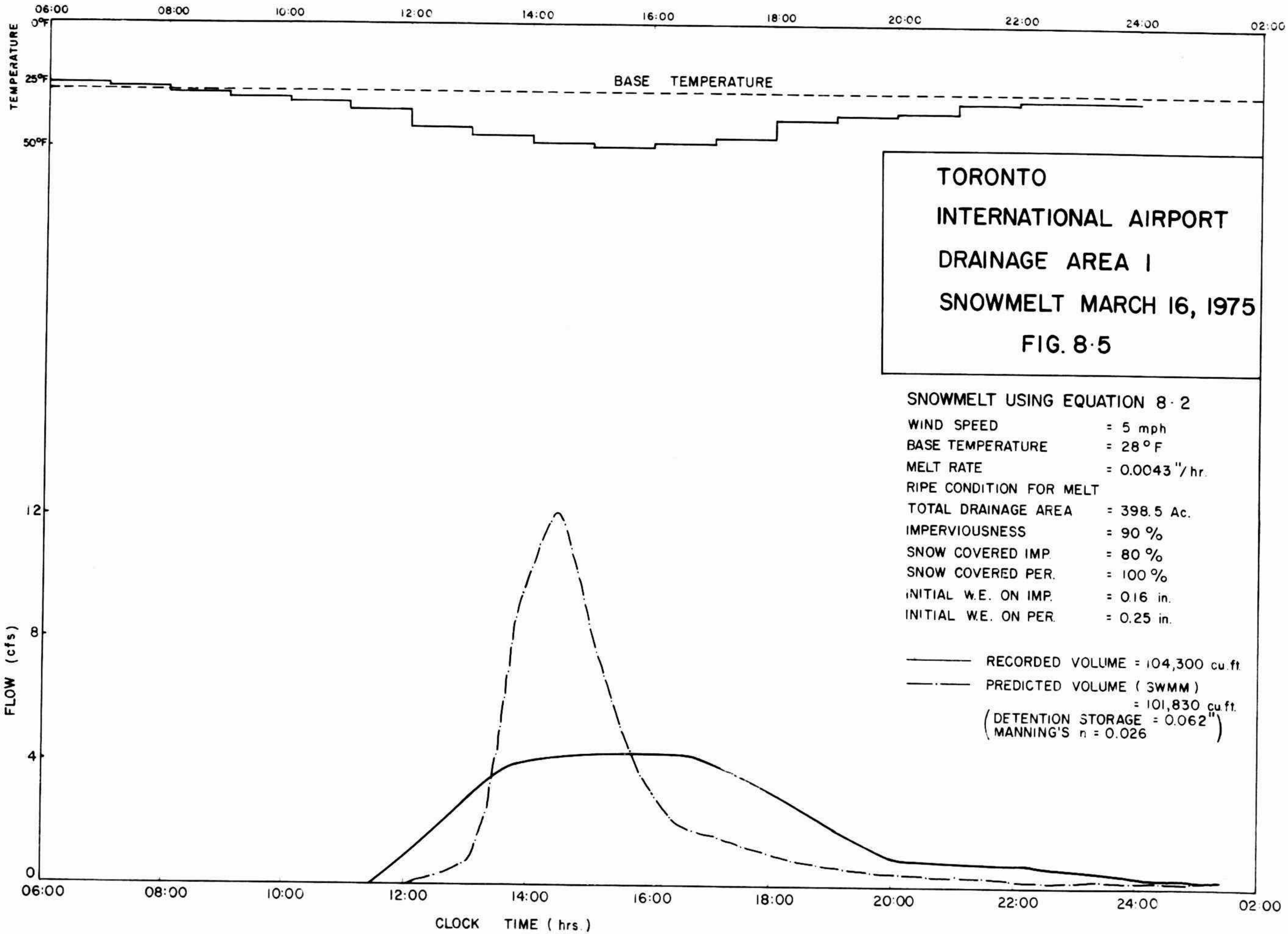




TESTING OF SNOWMELT QUANTITY MODEL ON THE BRUCEWOOD CATCHMENT USING HYPOTHETICAL DATA
FIG. 8.3
SNOWMELT USING EQUATION 8·3
WIND SPEED = 10 MPH
RIPE CONDITION FOR MELT
SNOW COVERED IMP = 100 %
SNOW COVERED PER = 100 %
INITIAL W·E ON IMP = 2.5 IN.
INITIAL W·E ON PER = 2.5 IN.
RUN 1 – RAIN ONLY
RUN 2 – RAINFALL AND SNOWMELT
RUN 3 – SNOWMELT ONLY
RAINFALL INTENSITY (IN/HR)
0.25
0.50
0.75
1.0
TEMPERATURE
°F
50° F
100° F
0
10
20
30
40
50
60
70
80
90
FLOW (cfs)
2
4
6
8
10
12
14
16
18
20
22
24
26
TIME (min.)
100
110
120
130
140
150
160
170
180
190
200

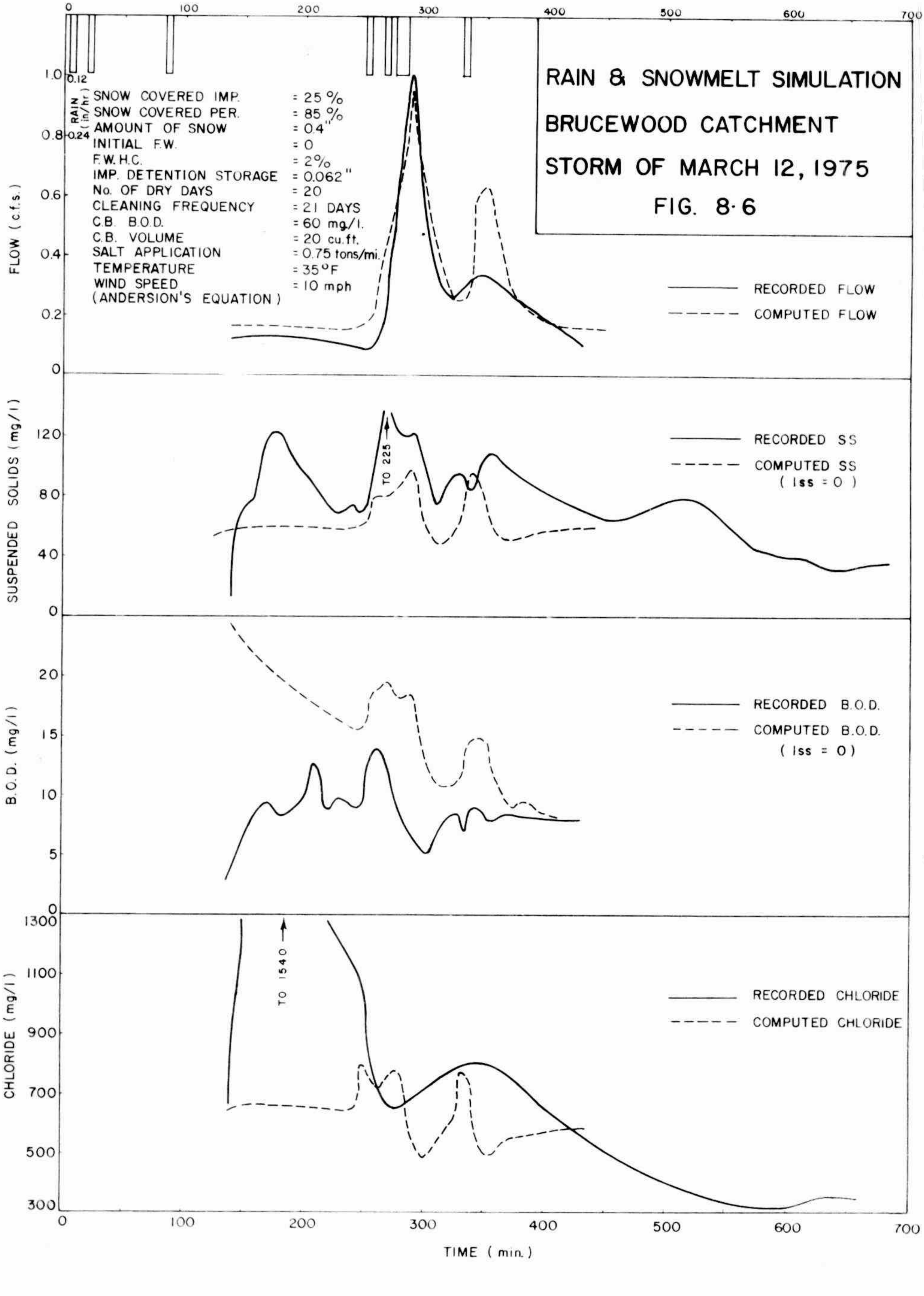


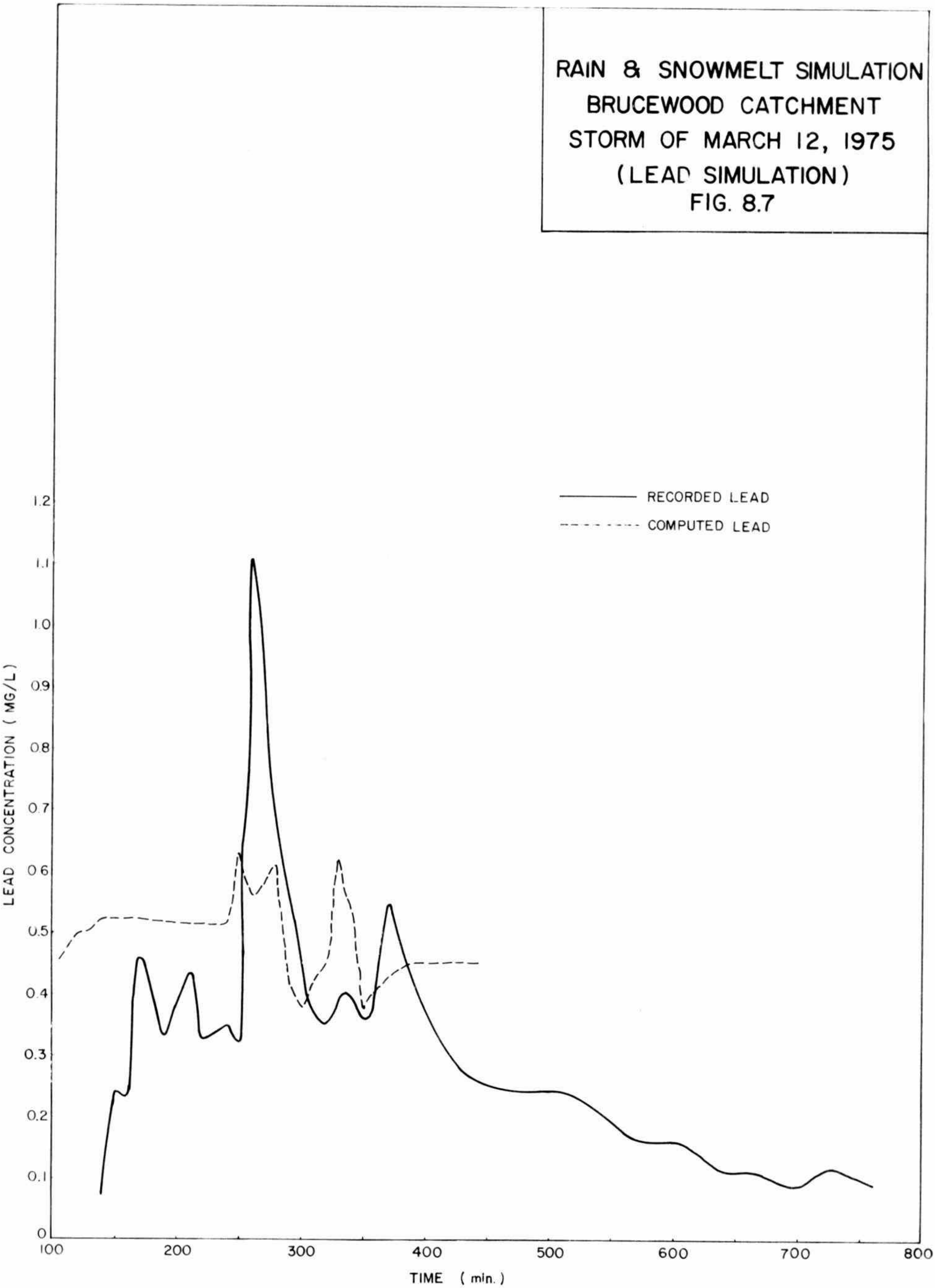
06:00
08:00
10:00
12:00
14:00
16:00
18:00
20:00
22:00
24:00
02:00
0°F
25°F
50°F
TEMPERATURE
BASE TEMPERATURE
TORONTO
INTERNATIONAL AIRPORT
DRAINAGE AREA I
SNOWMELT MARCH 15, 1975
FIG. 8·4
SNOWMELT USING EQUATION 8·2
BASE TEMPERATURE = 28 °F
MELT RATE = 0.0043 "/ hr.
RIPE CONDITION FOR MELT
TOTAL DRAINAGE AREA = 398.5 Ac.
IMPERVIOUSNESS = 90 %
SNOW COVERED IMP. = 80 %
SNOW COVERED PER. = 100 %
INITIAL W.E. ON IMP. = 0.4 in.
INITIAL W.E. ON PER. = 0.5 in.
RECORDED VOLUME = 143,100 cu. ft.
PREDICTED VOLUME (SWMM) = 171,807 cu. ft.
DETENTION STORAGE = 0.062"
MANNING'S n = 0.026
12
8
4
0
FLOW (cfs)
CLOCK TIME (hrs.)



TEMPERATURE
0°F
25°F
50°F
BASE TEMPERATURE
FLOW (cfs)
0
4
8
12
06:00
08:00
10:00
12:00
14:00
16:00
18:00
20:00
22:00
24:00
02:00
CLOCK TIME (hrs.)
TORONTO
INTERNATIONAL AIRPORT
DRAINAGE AREA I
SNOWMELT MARCH 16, 1975
SNOWMELT USING EQUATION 8·2
WIND SPEED = 5 mph
BASE TEMPERATURE = 28 °F
MELT RATE = 0.0043 "/hr.
RIPE CONDITION FOR MELT
TOTAL DRAINAGE AREA = 398.5 Ac.
IMPERVIOUSNESS = 90 %
SNOW COVERED IMP. = 80 %
SNOW COVERED PER. = 100 %
INITIAL W.E. ON IMP. = 0.16 in.
INITIAL W.E. ON PER. = 0.25 in.
RECORDED VOLUME = 104,300 cu.ft.
PREDICTED VOLUME (SWMM) = 101,830 cu.ft.
(DETENTION STORAGE = 0.062" MANNING'S n = 0.026)

FIG. 8·5


RAIN & SNOWMELT SIMULATION
BRUCEWOOD CATCHMENT
STORM OF MARCH 12, 1975
FIG. 8·6
RAIN (in/hr)
0.12
0.24
SNOW COVERED IMP. = 25 %
SNOW COVERED PER. = 85 %
AMOUNT OF SNOW = 0.4"
INITIAL F.W. = 0
F.W.H.C. = 2%
IMP. DETENTION STORAGE = 0.062"
No. OF DRY DAYS = 20
CLEANING FREQUENCY = 21 DAYS
C.B. B.O.D. = 60 mg/l.
C.B. VOLUME = 20 cu.ft.
SALT APPLICATION = 0.75 tons/mi.
TEMPERATURE = 35°F
WIND SPEED = 10 mph
(ANDERSION'S EQUATION)
FLOW (c.f.s.)
RECORDED FLOW
COMPUTED FLOW
SUSPENDED SOLIDS (mg/l)
TO 225
RECORDED SS
COMPUTED SS (Iss = 0)
B.O.D. (mg/l)
RECORDED B.O.D.
COMPUTED B.O.D. (Iss = 0)
CHLORIDE (mg/l)
TO 1540
RECORDED CHLORIDE
COMPUTED CHLORIDE
TIME (min.)


RAIN & SNOWMELT SIMULATION
BRUCEWOOD CATCHMENT
STORM OF MARCH 12, 1975
(LEAD SIMULATION)
FIG. 8.7
RECORDED LEAD
COMPUTED LEAD
LEAD CONCENTRATION (MG/L)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
100
200
300
400
500
600
700
800
TIME (min.)

EQUIVALENT CATCHMENTS (LUMPING)

CHAPTER 9

CHAPTER 9
EQUIVALENT CATCHMENTS (LUMPING)

9.1 GENERAL

The costs of setting up and running a SWMM simulation are largely determined by the level of discretization used for a particular catchment. The number of subcatchments into which the catchment is divided should depend on the information the user requires from the simulation. If flow and quality information describing conditions in each of the major conduits of a system is of interest, then it is necessary to employ a fine discretization, defining the subcatchments tributary to each of the conduits modelled. Conversely, if the object is merely to simulate the outflow from the entire catchment, it may be desirable to reduce the time and effort required for simulation through use of a few larger subcatchments and conduits (employing a coarse discretization). The purpose of this part of the study was to investigate the level of detail necessary to adequately represent an urban watershed and to illustrate the effects of reducing the number of subcatchments on the accuracy of runoff simulation.

A methodology was defined to achieve a representative equivalent catchment from theoretical considerations. Verification of the procedures involved a series of applications on both hypothetical and real areas. The results of the simplified methodology were compared with those of detailed simulations. The term "lumped" is used frequently in this chapter to describe simulations based on equivalent subcatchments. A lumped model may be defined as one which does not take into account the spatial variation of the parameters that determine hydrologic processes, (for instance when a single catchment is used to describe the entire runoff producing area).

9.2 DISCUSSION OF PREVIOUS WORK

A review of previous work relating to the application of SWMM revealed that the concept of lumped simulation has been mentioned by several authors.

Metcalf Eddy et al [1] obtained a reasonably good simulation with a 5 subcatchment discretization of the Northwood area, compared to a more detailed 12 subcatchment discretization. It was concluded that the overall accuracy of the simulation is reduced with simplification of the input data. Heeps and Mein [2] observed an increase in peak flow of about 20% accompanied by an advance in both the rising and falling limbs of the hydrograph, as the number of subcatchments was reduced. This was attributed to the elimination of the conduit storage available in a finely discretized system and it was concluded the degree of discretization has a significant influence on the simulated hydrograph. Shubinski and Roesner [3] discussed the simplified application of SWMM to large areas and concluded that this was feasible, although no comparative simulations were conducted. Jewell et al [4] observed an increase in peak flow and time to peak as the level of discretization at the Greenfield test basin was decreased. It was concluded that the model is not extremely sensitive to variations in subcatchment size or to the location of gutters and pipes.

This review of previous work indicates that no systematic study of alternate methods of aggregation has been conducted and that the use of longer time intervals in the rainfall hyetograph has not been considered.

9.3 BASIC CONCEPTS

9.3.1 *Overland Flow*

Overland flow computations are made in the RUNOFF block of the SWMM. A provision is made in this block for minor flow routing computations in gutters and small pipes. The storage available in these is relatively small and consequently their effects were not included in the lumped applications

(except in the smallest study area). The discussion of routing in the TRANSPORT block appears in the next section.

The overland flow hydrograph for a subcatchment is determined in SWMM over a series of time steps during each of which the outflow rate is averaged. This rate is computed from Manning's equation, with the average ground slope of the subcatchment taken as the slope of the hydraulic grade. The excess depth of water (following subtractions for infiltration and detention storage) is assumed constant over the flow plane for a given time step and is used as the hydraulic radius for the flow plane. The following equations are used:

$$V = \frac{1.49}{n} \cdot De^{2/3} \cdot S^{1/2} \qquad (9.1)$$

$$Qw = V.W.De \qquad (9.2)$$

where V = velocity
n = Manning's roughness
De = excess water depth overflow plane
W = width of the flow plane
S = slope of the flow plane

The equation of continuity is solved for the depth of water remaining over the flow plane at the end of each time step.

$$De(2) = De(1) - \frac{(Qw)}{A} .\Delta t \qquad (9.3)$$

where A = subcatchment area
Qw = outflow rate during the time step
Δt = duration of time step
De(1) = excess intermediate depth after accounting for infiltration
De(2) = excess water depth resulting from rainfall, infiltration, and outflow during the time step

The parameters controlling the overland flow from pervious or impervious areas are:

- ground slope
- Manning's roughness
- retention depth
- infiltration
- width of overland flow

Of these parameters all except "width" are assumed to be spatially constant over the area of the subcatchment. Consequently, when aggregating subcatchments, it is necessary to assign these spatially constant parameters values representative of the aggregated area as a whole. The "width" parameter, however, is defined as that dimension of a subcatchment across which overland flow travels when draining to the inlet manhole or gutter draining that area (See Figure 9-1). It is important to remember that this is not necessarily the physical width of the subcatchment itself. The "width", therefore is a measure of the ease with which surface runoff may discharge from the subcatchment, and is a function of the drainage density and number of inlets. From equation (9.2) it may be seen that the rate of outflow in any time interval is directly proportional to the "width". A reduction in "width" will result in a reduced surface runoff and hence cause a greater amount of excess water to be built up on the flow plane in response to a given rainfall input. Conversely, an increase in "width" will allow water to drain more rapidly from the flow plane, and less surface storage will result. This effect can be employed to attenuate the overland flow hydrograph and to compensate for conduit storage which is neglected in the lumping process. Figure 9-2 shows the effect of variations in the "width" on the overland flow hydrograph for an 11 acre test area. If the correct width of 1414 feet is reduced, the hydrograph is delayed and the peak reduced due to increased artificial storage. Conversely, if the width is increased the hydrograph is advanced and the peak flow increased.

The above discussion leads to the conclusion that if it is desired to combine several smaller subcatchments into a single "lumped" catchment with the same surface runoff hydrograph, it would be logical to use average catchment values for ground slope, roughness, retention depth, and infiltration, but to use the sum of the individual subcatchment "widths" as the width of the "lumped" catchment.

9.3.2 *Routing*

In order to maintain an accurate simulation of flow through a sewer system it is necessary to specify an equivalent transport system, which results in the same attenuation and delay of the inlet hydrographs as the actual system. A certain amount of conduit or channel aggregation is generally required in most detailed simulations, due to computational limitations of the SWMM. The conduit dimension and slope are the most important properties involved in the SWMM routing computations. The general procedure recommended in the EPA SWMM User's Manual, is to replace several of the smaller conduits with an equivalent element having an average size, slope, length, and roughness. It may be anticipated that as fewer subcatchments are employed, more small conduits must be neglected with a consequent reduction of conduit storage in the routing computations. Such a reduction will result in higher peak flows and a shorter time to peak in the outlet hydrograph. Two methods may be employed to introduce additional storage into the simulation and compensate for the neglected smaller conduits. The length of the aggregated conduit may be increased to add more conduit storage to the system, or the overland flow "width" of the equivalent or "lumped" catchment can be reduced in order to introduce the desired attenuation and time delay into the computations. It is important also, to consider the relative effects of both the overland flow routing and sewer system routing. In many small urban catchments, the sewer routing effects are less significant than overland flow effects, and hence the outflow hydrograph will not be very sensitive to changes in the sewer system.

9.4 PROCEDURES AND ALTERNATIVE METHODS FOR SIMPLIFIED SIMULATION

A lumped simulation may involve a single equivalent catchment, or in the case of a number of distinctly different drainage sub-areas, may utilize several aggregated areas. This latter case may be necessary where upstream controls, storage or diversions necessitate special consideration and the use of both overland flow and sewer routing computations. However, in many cases it would be possible to use the RUNOFF block alone to compute an outlet hydrograph. By reducing the lumped catchment overland flow width, an extra measure of

routing effect can be introduced to make up for the sewer routing effects in the TRANSPORT block. In order to assess the utility of these concepts, a series of procedures were devised for performing lumped simulations. Detailed simulations for different areas were available against which the results of lumped simulations could be compared. Lumped simulations were prepared according to the following guidelines.

9.4.1 *Simplified Simulation with RUNOFF and TRANSPORT*

(a) Following detailed simulations, the individual subcatchments were aggregated to provide different levels of discretization (i.e. detailed, 5 and 1 catchments). Parameters describing surface characteristics (infiltration, detention depth, ground slope, Manning's 'n') were computed for each equivalent catchment based upon subcatchment areas.

(b) The overland flow width for each aggregated catchment was assumed to be the sum of the widths of the individual constituent subcatchments.

(c) The equivalent transport system was established by computation of the weighted (by length) average dimensions and slopes of the detailed constituent elements. Where a single aggregate catchment was modelled, a corresponding single equivalent transport element, representative of the real trunk system was employed. The length of this element was approximated to the length of the actual major trunk conduit.

9.4.2 *Simplified Simulation with RUNOFF Block Only*

(a) A single equivalent catchment was aggregated from the detailed discretization as before.

(b) Several equivalent widths were employed for testing purposes. These ranged from the sum of the subcatchment widths to the simple dimension of the lumped catchment, transverse to the overall sewer flow.

(c) No transport element was used.

These procedures are schematically illustrated in Figure 9-3. Case 1 indicates a small area discretized into 4 subcatchments. Overland flow is assumed to occur perpendicular to the drainage conduits and the corresponding "widths" are shown as broken lines. The "width" would be determined for each subcatchment according to the User's Manual. The sum of the individual overland flow hydrographs is represented in (a), the total surface runoff hydrograph having a peak of Q_R. Subsequent routing of this hydrograph through the TRANSPORT system results in an outflow hydrograph having a peak, Q_{OUT}. These have been given peak values of 60 cfs and 35 cfs, respectively, for purposes of illustration.

Case 2 illustrates the lumping of the individual areas to form a single equivalent catchment. In this example, the equivalent surface parameters are computed using an area weighted average and the width is assumed to be the sum of the four constituent widths in order to preserve the overland flow hydrograph, and is shown as the broken line. The original transport elements have been replaced by an equivalent conduit, the length of which is approximately that of the major trunk in the finely discretized system. The results of the lumped simulation are similar to those of the detailed simulation (Case 1).

Case 3 illustrates a lumped simulation using only the RUNOFF block. The overland flow width is reduced to account for the conduit storage neglected by not using the TRANSPORT routines. The reduced width is indicated by the broken line on the diagram. The diminished width causes the overland flow hydrograph to be retarded and attenuated. Selection of an appropriate width results in a RUNOFF block surface hydrograph which is very similar to that of the TRANSPORT block in Case 1 and 2.

The simplifying procedures outlined above were applied on several test areas, and are discussed in the following sections. The lumped catchment parameters were based upon detailed simulations. In practice, the lumped parameters would be developed directly from basic maps and plans to represent the entire catchment, and the lumped "width" would be an additional calibration parameter.

9.5 APPLICATION OF LUMPING CONCEPTS

The simplifying procedures discussed above were applied on hypothetical and real test areas. The test areas (Table 9.1) ranged in size from 6 to 2330 acres. Detailed simulations were performed for each area and subsequently lumped simulations were performed and the results compared to those from the detailed simulations.

TABLE 9.1

TEST AREAS FOR STUDIES OF SIMPLIFIED PROCEDURES

Area	Level of Discretization	Number of Subcatchments	Number of Transport Routing Elements
Hypothetical (643 acres)	detailed simulation	37	38
	coarse simulation	5	30
	single catchment	1	1
	single catchment	1	0
West Toronto (2330 acres)	detailed simulation	45	73
	single catchment	1	1
	single catchment	1	0
Bannatyne (542 acres)	detailed simulation	41	62
	coarse simulation	3	3
	single catchment	1	0
Small Winnipeg Area (6 acres)	detailed simulation	6	6
	single catchment	1	0

9.5.1 *Quantity Effects*

For each of the test areas a number of storm hyetographs were selected covering a range of intensities and durations in order to assess the validity of simulation over a range of hydrologic occurrences. A series of synthetic storms were applied over the hypothetical test areas, following the work of Brandstetter [5]. Table 9.2 summarizes the storm patterns used in each area.

TABLE 9.2

STORM PATTERNS FOR TEST APPLICATIONS

Area	Peak Intensity in inches per hour	Duration in hours
Hypothetical	3.67	1 (triangular shape)
	1.92	2 (triangular shape)
	.98	4 (triangular shape)
	1.00	2 (rectangular shape)
	.50	4 (rectangular shape)
West Toronto	.84	2.1 (September 23,1973)
	2.16	.4 (October 2,1973)
	2.28	1.0 (May 10, 1973)
Bannatyne	.33	3.3 (June 19, 1971)
	1.20	1.5 (September 5,1971)
Small Winnipeg Area	5.65	1.0 (5 yr. Design Storm)

9.5.2 *Hypothetical Test Areas*

The hypothetical test area was designed to have uniform properties in subcatchments in order to facilitate the process of aggregation. Three levels of discretization were employed. These are indicated in Figure 9-4. Each diagram depicts the complete subcatchment and transport system used in the lumped simulations. Five synthetic storms were applied to the three discretizations of the hypothetical area. Both RUNOFF and TRANSPORT blocks were employed in these simulations. At both levels of lumped discretization the total overland flow width was 79000 ft., the same as in the detailed case. The equivalent transport elements employed in the simplified simulations were determined from an aggregation of the network used in the detailed system.

The results of the individual simulations for the 5 rainfall patterns are shown in Figures 9-5 to 9-9. The hydrographs computed by the lumped model are very similar to those of the detailed simulations. A trend of increasing peak flow with decreasing number of equivalent subcatchments was observed. The greatest increase in peak flow occurred for the intense 1-hour storm, as indicated in Table 9.3.

TABLE 9.3

Rainstorm	Subcatchments	Peak Flow in (CFS)	Total Runoff Volume (ft.^{3})
1 hr.	37	1104	3.00×10^6
	5	1163	3.00×10^6
	1	1176	3.00×10^6
2 hrs.	37	633	2.75×10^6
	5	629	2.74×10^6
	1	647	2.74×10^6
4 hrs.	37	305	2.36×10^6
	5	306	2.36×10^6
	1	310	2.36×10^6
2hrs. rect.	37	403	2.62×10^6
	5	404	2.62×10^5
	1	409	2.62×10^6
4hrs. rect.	37	163	2.28×10^6
	5	162	2.28×10^6
	1	163	2.28×10^6

The increase in peak flow is less significant in the case of storms of longer duration and lower intensity. This is because as the storm duration increases, steady state conditions occur, in which the outflow rate is equal to the applied rainfall minus losses. In the 4 hour storm simulations, a steady state of runoff from the lumped catchment is observed. The equivalent transport system, provides a fairly good approximation of the detailed system in view of the relatively small increases in peak flow. These could be eliminated by an appropriate increase in length of the equivalent transport system. Such a measure could be employed to remedy the minor advance of the entire outflow hydrograph in time as fewer subcatchments are employed. This advance, which is also attributable to a loss of conduit storage, was limited to 10 minutes in the worst case. This would probably not be a significant factor in most practical modelling applications.

Additional simplified simulations using only the RUNOFF block were conducted in order to investigate the complete elimination of the TRANSPORT block. These simulations all used the 2-hour triangular storm. Results were again compared with those of the detailed discretization involving 37 subcatchments. No TRANSPORT elements were used in these simulations. The overland flow "width" was reduced to induce more surface storage to compensate for the exclusion of all conduit storage. Several simulations were necessary to determine the optimum "width". The results of these simulations are summarized in Table 9.4, and the complete hydrographs are shown in Figure 9-10.

TABLE 9.4

SIMPLIFIED SIMULATION USING RUNOFF BLOCK ONLY

No. of Subcatchments	Total Overland Flow Width All Sub-catchments (ft.)	Peak Flow In cfs	Total Runoff Volume (ft.^{3})	Total Surface Storage at end of Simulation (ft.^{3})
37	79,000	633*	2.74×10^6	$.569 \times 10^5$
1	20,000	542	2.44×10^6	$.731 \times 10^5$
1	39,000	608	2.58×10^6	$.616 \times 10^5$
1	45,000	622	2.61×10^6	$.602 \times 10^5$
1	56,000	640	2.66×10^6	$.587 \times 10^5$

*(Detailed simulation using RUNOFF and TRANSPORT blocks).

These results indicate that in this case, a reduction in overland flow width from 79,000 ft. to 56,000 ft. is necessary to compensate for the conduit storage neglected through elimination of the TRANSPORT block. There is no direct relationship between conduit storage and surface storage, and consequently it is not possible to determine beforehand the exact value of overland flow "width" required for an accurate simulation. The peak flow obtained in these simulations is relatively insensitive to the reductions in "width", over much of the range. A 75% reduction in "width" to 20,000 ft. results in a peak flow reduction of only 15%. The total length of trunk conduit in the hypothetical area was 20,000 ft. The overland "width" which produced the best simulation was 2.8 times the length of main drainage conduit in the real system.

These results also illustrate the variation of surface storage with width. As the width is reduced, more water is left on the overland flow plane at the end of each simulation with a consequent reduction in total surface runoff. If the simulation were allowed to run on for a much longer time period, most of this volume would be recovered.

9.5.3 *Real Test Areas*

The real test areas provided a more realistic set of parameters with which to test the lumping procedures.

9.5.3.1 *West Toronto*

Three storms were selected for the simplified simulations for this area. Initially a detailed simulation employing 45 subcatchments and 73 conduits (in TRANSPORT) was conducted for each event. Imperviousness and ground slope were measured from maps and plans of each subarea, while default values were assumed for infiltration, detention and roughness. Equivalent parameters were then computed to allow the entire 2330 acre area to be modelled as a single catchment. The overland flow width was taken as the sum of 45 individual widths and the equivalent transport element was aggregated from the main trunk sewers in the detailed system, which consists of five major trunks joining approximately 1500 feet from the outlet. The largest of these is of 111" diameter leading to a 96" conduit which approximately bisects the entire catchment, and is some 9,000 feet long to the junction with a 75" and a 51" pipe. The dimensions of the equivalent transport element selected to represent this large trunk sewer, were determined from weighted average sizes and slopes of the real trunk. As a first approximation, the actual length of 9,000 feet was selected for the equivalent element. The results of the simplified simulations are shown in Table 9.5 and in Figures 9-11 to 9-13.

TABLE 9.5

WEST TORONTO SIMPLIFIED SIMULATION

(RUNOFF & TRANSPORT)

Storm	No. of Sub-catchments	Peak Flow in cfs	Total Runoff Volume (ft.^{3})	Total Infil-tration (ft.^{3})	Total Surface Storage at end of Storm (ft.^{3})
Sept.23,1973	45	366	$14.59x10^5$	$14.45x10^5$	$2.19x10^5$
Sept.23,1973	1	369	$14.44x10^5$	$14.45x10^5$	$2.32x10^5$
Oct. 2,1973	45	421	$8.57x10^5$	$9.37x10^5$	$2.25x10^5$
Oct. 2,1973	1	455	$8.33x10^5$	$9.37x10^5$	$2.47x10^5$
May 10,1973	45	710	$27.16x10^5$	$25.4x10^5$	$2.32x10^5$
May 10,1973	1	947	$26.78x10^5$	$25.4x10^5$	$2.64x10^5$

The results indicate a generally good agreement between detailed and lumped simulations. The total runoff volume computed for the single catchment is within 3% of the detailed result. Peak flow rates show a minor increase for the simplified simulations for the storms of Sept. 23, and Oct. 2. Initial simulations (using an equivalent transport element of 9000 feet) produced a slightly greater increase in peak flow, and consequently the length of the element was increased by 10%. The accuracy of the simulation of May 10, 1973 appears to depart substantially from that obtained in the first two simulations. Figure 9-13 shows the results of the simulations for this storm. In the detailed simulation many of the upstream pipes were surcharged. This resulted in a truncation of the peak flow as indicated, (the SWMM simply stores flows in excess of pipe capacities). Since only the major trunk line was represented in the simplified simulation, there was no surcharging and the outflow was greater. Nonetheless similar values of total runoff volume were computed at both levels of discretization.

As observed in the hypothetical area simulations, a slight advance of the outflow hydrograph occurred. This is attributable to conduit storage which is neglected in the lumped simulations.

Additional simulations were performed using the RUNOFF block alone. A reduction of the overland flow width was used to compensate for the loss of conduit storage. Several simulations were done for the Sept. 23 storm to find optimum value of equivalent width, and the results are shown in Table 9.6 and Figure 9-14.

TABLE 9.6

WEST TORONTO SIMPLIFIED SIMULATION

(RUNOFF Only)

1973 Storm	No. of Sub-catchments	Overland Flow Width All Subc's (ft.)	Peak Flow in cfs	Total Runoff Volume (ft^3)	Total Infil-tration (ft^3)	Total Surface Storage at end of Storm ($ft.^3$)
Sept.23	45	200,000	366*	14.59×10^5	14.59×10^5	2.19×10^5
Sept.23	1	130,000	424	14.57×10^5	14.45×10^5	2.23×10^5
Sept.23	1	40,000	319	13.99×10^5	14.45×10^5	2.66×10^5
Sept.23	1	60,000	373	14.29×10^5	14.45×10^5	2.44×10^5

(*detailed simulation using RUNOFF and TRANSPORT blocks)

These results emphasize the sensitivity of the results of the single equivalent catchments simulations to variations in overland flow width. A reduction in "width" to 130,000 feet was insufficient to account for the neglected conduit storage and the lumped peak flow was too high. Two further simulations were required to achieve a close agreement. A "width" of 60,000 feet, or roughly one third of the detailed simulation overland flow "widths" produced accepatable results. This was 1.7 times the length of the main trunk conduit in the actual system. The total runoff volume decreased with decreasing width, which is to be expected since additional surface storage has been introduced. This is confirmed by the fact that the total surface storage remaining at the end of the simulation increased as the width was reduced. The total infiltration was about 1% less for the lumped simulations, indicating that the aggregated catchment parameters closely represented those used in the detailed discretization.

9.5.3.2 *Winnipeg Areas*

Simplified modelling procedures were applied to two test areas in Winnipeg: Bannatyne (542 acres), and a small subsection of a neighbouring drainage area (6 acres). Simulations were performed to investigate the effect of lumping on smaller catchments in which conduit routing effects were relatively minor. Detailed simulations were conducted for both areas, and lumped simulations employing both the RUNOFF and TRANSPORT blocks were subsequently performed. Equivalent parameters were computed for the single catchment used to represent each test area. The TRANSPORT block was used for flow routing for the Bannatyne district, while the GUTTER routine of the RUNOFF block was employed for this function in the small, 6 acre area. The single equivalent transport element was based on the length and size of the main drainage conduit in each area. The results of the lumped simulations are shown in Table 9.7, and on Figures 9-15, 9-16 and 9-17.

TABLE 9.7

WINNIPEG SIMPLIFIED SIMULATION
(RUNOFF + TRANSPORT)

Area	Storm	No. of Sub-catchments	Peak Flow in cfs	Total Runoff Volume ($ft.^3$)	Total Infil-tration ($ft.^3$)	Total Surface Storage at end of Storm ($ft.^3$)
Bannatyne	June 19,1971	41	31	2.85×10^5	5.65×10^5	$.34 \times 10^5$
Bannatyne	June 19,1971	1	36	2.84×10^5	5.65×10^5	$.34 \times 10^5$
Bannatyne	Sept. 5,1971	41	97	3.71×10^5	7.16×10^5	$.34 \times 10^5$
Bannatyne	Sept. 5,1971	1	93	4.10×10^5	7.16×10^5	$.38 \times 10^5$
Small Test Area	5 yr Design Storm	6	9	$.18 \times 10^5$	$.26 \times 10^5$	$.24 \times 10^5$
Small Test Area	5 yr Design Storm	1	10	$.19 \times 10^5$	$.24 \times 10^5$	$.24 \times 10^5$

These results indicate a good agreement between the simplified and detailed simulations in total runoff volume computed. As experienced in previous simulations there was an increase in peak flow in the lumped simulations, primarily due to neglected conduit storage.

9.5.3.3 *Quality Effects*

In the SWMM, storm water quality is computed from an exponential decay surface washoff equation and simple mass balances to establish concentrations within the sewer system. The amount of pollutant washoff is a function of the rate of stormwater runoff, and the initial amount of accumulated surface pollutants. Assuming the correct rate of stormwater runoff is maintained for the equivalent catchment, the correct pollutant washoff rate may be maintained by providing the weighted average pollutant loading rate, and the aggregated gutter length in the lumped catchment simulation. This would result in the same total amount of accumulated dust and dirt at the start of the storm as in the detailed simulation.

Sensitivity analyses discussed in Chapter 5 have shown that pollutant decay within the sewer system is not significant. Consequently outlet pollutographs computed for both detailed and simplified simulations should be similar, even though the transport system maybe reduced or neglected in the coarse discretization.

Simulations of the hypothetical area including quality computations were conducted for two storms. Aggregated quality parameters were the dust and dirt loading rates and gutter lengths. These dust and dirt loading rates were computed using a weighted average rate, based upon the area in each of the 5 land use categories. All 5 categories were modelled in the detailed simulation. An immediate problem was encountered in that these loading rates, which are a function of land use, are fixed internally in the SWMM and only a single land use may be assigned to each subcatchment. Consequently there was no means to supply the appropriate loading rate for the equivalent catchment. The SWMM was modified to permit direct card input of the desired loading rates for each land use. The aggregate catchment was arbitrarily assigned a single land use, and the average loading rate specified. The total gutter length was maintained

in the lumped simulation ensuring the same initial surface pollutant load as in the detailed simulation. Surface runoff quality is also affected by the volume of water stored in catchbasins. The same number and volume of catchbasins was specified in the lumped catchment as in the constituent subcatchments. Lumped simulations were performed for two synthetic storm patterns. Table 9.8 summarizes the results of these simulations. Default values were used to describe runoff quality parameters in the detailed simulations.

TABLE 9.8

SIMPLIFIED QUALITY SIMULATION - HYPOTHETICAL AREA

Storm	No. of Sub-catchments	Total BOD Computed in lbs	Total SS Computed in lbs
1	37	4,484	62,144
1	1	4,482	62,142
2	37	5,565	85,122
2	1	5,590	85,190

These results show a close agreement of total BOD and SS washoff at the different levels of discretization. A small increase in peak concentrations, and a slight advance of the pollutographs was observed in the simplified simulations. This corresponded to fluctuations in the outflow hydrograph noted in the previous sections.

The foregoing discussion shows that lumping precedures may be applied to the surface runoff quality computations without incurring significant discrepancies in the resulting pollutographs. However other factors also influence the storm water quality computations. Dry weather flow flushing of sewer deposits are modelled in the TRANSPORT block. The deposition and scour effects which determine the first flush of pollutants depend upon dry weather flow velocities,

conduit slopes, and conduit sizes. Adjustment of the equivalent transport element of a single subcatchment simulation in order to duplicate the detailed simulation first flush effect appears to be unfeasible, since such action would affect the outflow hydrograph. The amount of solids deposited in conduits at the start of a storm is sensitive to conduit slope. If only the RUNOFF block is utilized, these dry weather flow effects cannot be modelled. Therefore it is felt that it is not possible to model dry weather flow quality effects in a lumped simulation.

9.6 TEMPORAL EFFECTS OF AGGREGATION

The successful use of the SWMM in a spatially aggregated mode questions the necessity for short interval rainfall data and short computational time steps. It is normal practice to match the input rainfall time step with the computational time step. The lumped SWMM no longer performs detailed routing in the sewer system and consequently longer time intervals might be employed for the input rainfall hyetograph if the appropriate RUNOFF and TRANSPORT routines prove stable. The potential benefits to be derived from use of a longer time step are; more readily available rainfall data (in hourly intervals), and the possibility of using the SWMM for continuous simulation based on hourly rainfall data.

9.6.1 *Quantity*

Simulations of the hypothetical test area were performed for time steps ranging from five minutes to one hour. The area was modelled using a single equivalent catchment including a single transport element. The GUTTER routing routine in the RUNOFF block was not utilized. Table 9.9 summarizes the results of this series of simulations, and the resulting hydrographs are shown on Figure 9-18.

TABLE 9.9

Storm	Sub-catchments	Time Step (min.)	Peak Flow (cfs)	Volume (ft.^{3})
4 hr. triangular	37	5	306	23.6×10^5
"	1	5	310	23.6×10^5
"	1	15	304	23.6×10^5
"	1	30	292	23.5×10^5
"	1	60	278	23.1×10^5

The results of these simulations indicate that larger time steps may be used to model the storms with a duration longer than the basin time of concentration, without a significant loss of accuracy. Both RUNOFF and TRANSPORT blocks were employed in these simulations, and no computational instability due to the use of 60-minute time steps was observed. In the event of instability in the TRANSPORT block, the RUNOFF block could be used in isolation for the simplified simulation. Table 9.9 indicates that the total runoff volume computed for the 5-minute and 60-minute time intervals is similar. The peak outflow rate displays a decrease of approximately 10% with the larger time step. This result is expected since use of larger time intervals attenuates the peaks in the hyetograph giving lower overall intensities.

In order to further investigate the effect of longer time steps over a range of hydrologic conditions, simulations were conducted using the 2-hour and 1-hour triangular synthetic storms. These durations were, respectively, equal to and less than the time of concentration of the hypothetical area. It might be expected that the shorter the storm duration, the greater would be error involved in using a 60-minute time step. Table 9.10 shows the results of these simulations.

TABLE 9.10

SIMULATION RESULTS USING HOURLY TIME STEPS

Storm	No. of Subcatchments	Time Step (Min.)	Peak Flow (cfs)	Volume of Runoff (Ft.^{3})
1 hr.	1	5	1176	2.99×10^7
1 hr.	1	60	438	2.90×10^6
2 hr.	1	5	647	2.74×10^6
2 hr.	1	60	414	2.33×10^6
4 hr.	1	5	310	2.36×10^6
4 hr.	1	60	278	2.31×10^6

The results demonstrate that for short duration storms, significant reductions in peak flow are caused by using hourly time steps. As the duration of the applied storm increases, the relative accuracy of the longer time interval simulations improves. Thus for the one hour storm, the peak flow is reduced by 62%, whereas for the 4-hour storm a reduction of only 10% was observed. The total runoff volume was reasonably close in all simulations. The maximum deviation in runoff volumes computed for short and long interval simulations was approximately 15%.

These results show that for long duration or uniform rainfall events, use of an hourly time step produces good results. For shorter and more sporadic events, however use of hourly steps will result in too much dampening in the rainfall-runoff process and can result in significant inaccuracy. This would be especially true for smaller catchments. Simpler models, such as STORM, are better suited to such a purpose. It is suggested that, in general, the maximum time steps employed in a continuous SWMM application should be limited to 20-30 minutes.

9.7 CONCLUSIONS

(a) An examination of the equations used in SWMM indicated the summation of the overland flow hydrographs generated for a number of individual subcatchments would be similar to that generated for an equivalent catchment based on the aggregated properties of those individual subcatchments.

(b) The principal effects of routing the overland flow through a detailed sewer system in the TRANSPORT block are an attenuation in the hydrograph peak and a retardation of the hydrograph due to conduit storage. These phenomena may be approximated in two ways:

(i) extension of the equivalent conduit element used in RUNOFF + TRANSPORT simulations.

(ii) reduction of the width for overland flow used in RUNOFF only simulations.

(c) Application of the simplified procedures on both hypothetical and real catchments substantiated the intuitive and theoretical basis for "lumping" and indicated that accuracy of simulation can be maintained by careful aggregation of subcatchment parameters.

(d) Practical lumped simulations may be conducted for a given catchment by using average values of the runoff parameters for the entire catchment. An initial estimate of the width of overland flow for the lumped catchment can be taken as twice the length of main trunk sewer in the area being modelled. This initial estimate of width is an approximation based on the values of 2.8 and 1.7 reported in Sections 9.5.2 and 9.5.3. Independent lumping studies [6] have recommended twice the longest dimension of the lumped catchment as an initial estimate of "width". Both of these

approximations may be applicable in any one case depending upon the extent of the drainage system and the relation between overland flow and channel routing effects. The "width" should be considered as an additional calibration parameter, and used to calibrate the lumped model with measurements, where possible.

(e) The lumped SWMM is a planning tool rather than a design or analysis tool. It should not be applied for design since it does not directly consider the effect of sewer surcharging or backwater, which may influence storm outflows.

(f) Simulations employing an increased time interval for hyetograph input and computational methods appear to be practical in cases where the time step used exceeds the time of concentration of the basin and where the computational stability is preserved. For short intense rainfall distributions peak flows will be inaccurate due to averaging of the rainfall hyetograph over a longer time interval, especially for small watersheds.

REFERENCE - CHAPTER 9

1. Metcalf & Eddy et al, "Storm Water Management Model", Volumes 1-4, U.S. Environmental Protection Agency document 11024DOC07/71, Washington D.C., July 1971.

2. Heeps, D.P., and Mein, R.G., "An Independent Evaluation of Three Urban Stormwater Models", Monash University Civil Engineering Research Report No. 4, Australia, 1973.

3. Shubinski, R.P., and Roesner, L.A., "Linked Process Routing Models", Symposium on models in urban hydrology, American Geophysical Union, Washington, D.C., April 16-20, 1973.

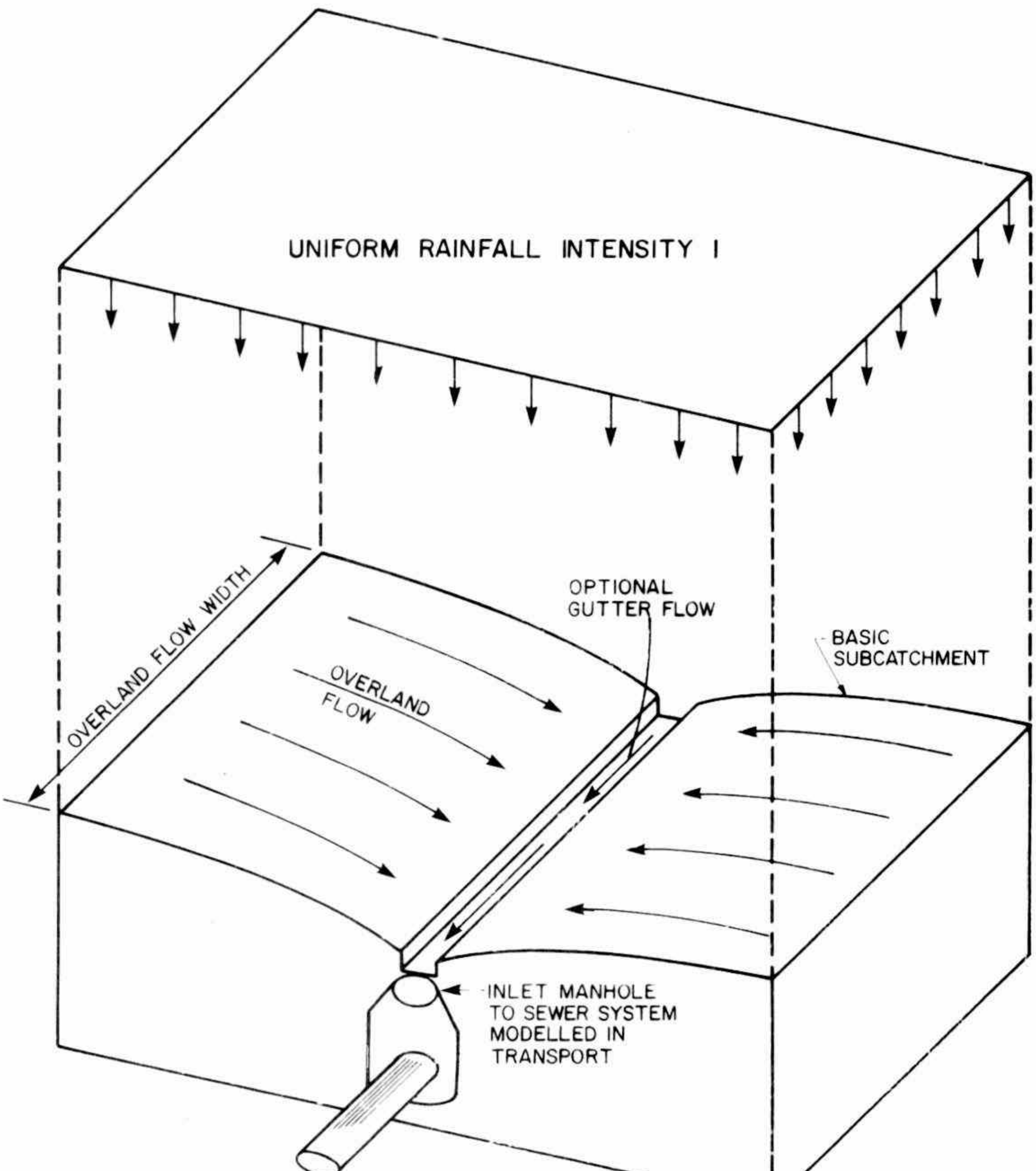
4. Jewell, T.K., et al, "Application and Testing of the EPA Storm Water Management Model to Greenfield, Massachusetts". In "Short Course on Applications of Storm Water Management Models" sponsored by the V.S.E.P.P. and the University of Massachusetts, August 19-23, 1974.

5. Brandstetter, A., " Assessment of Mathematical Models for Storm and Combined Sewer Management", Preliminary report, Office of Research and Development, U.S. Environmental Agency, Cincinnatti, 1975.

6. Smith, G.F., "Adaptation of the EPA Storm Water Management Model for use in Preliminary Planning for Control of Urban Storm Runoff", Unpublished M.Eng. thesis, Department of Environmental Engineering Sciences, University of Florida, June, 1975.

SCHEMATIC SWMM SURFACE DRAINAGE ARRANGEMENT

FIG. 9.1

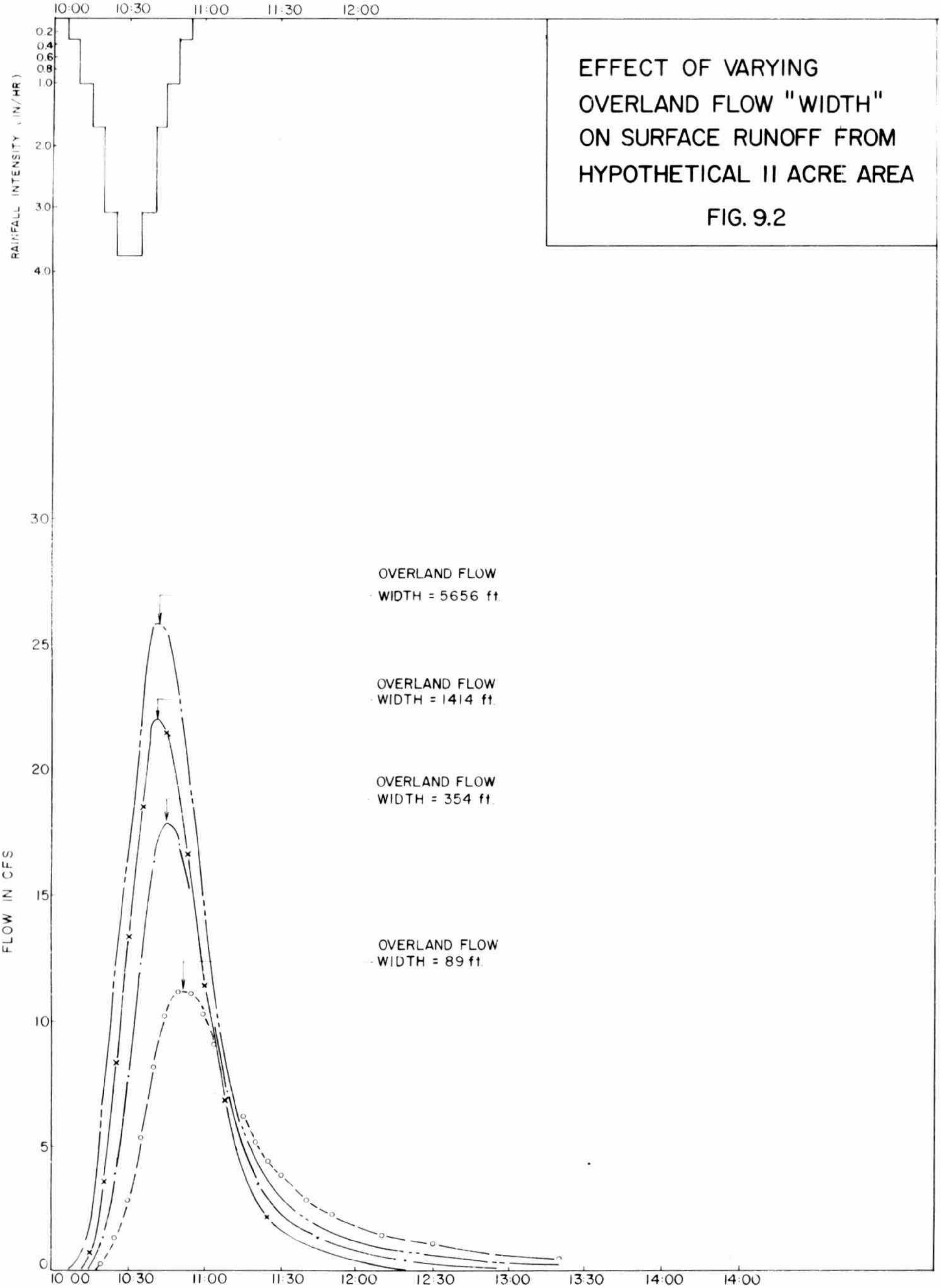


q_L = RATE OF OVERLAND FLOW / UNIT WIDTH

W = 2L = TOTAL WIDTH OF OVERLAND FLOW

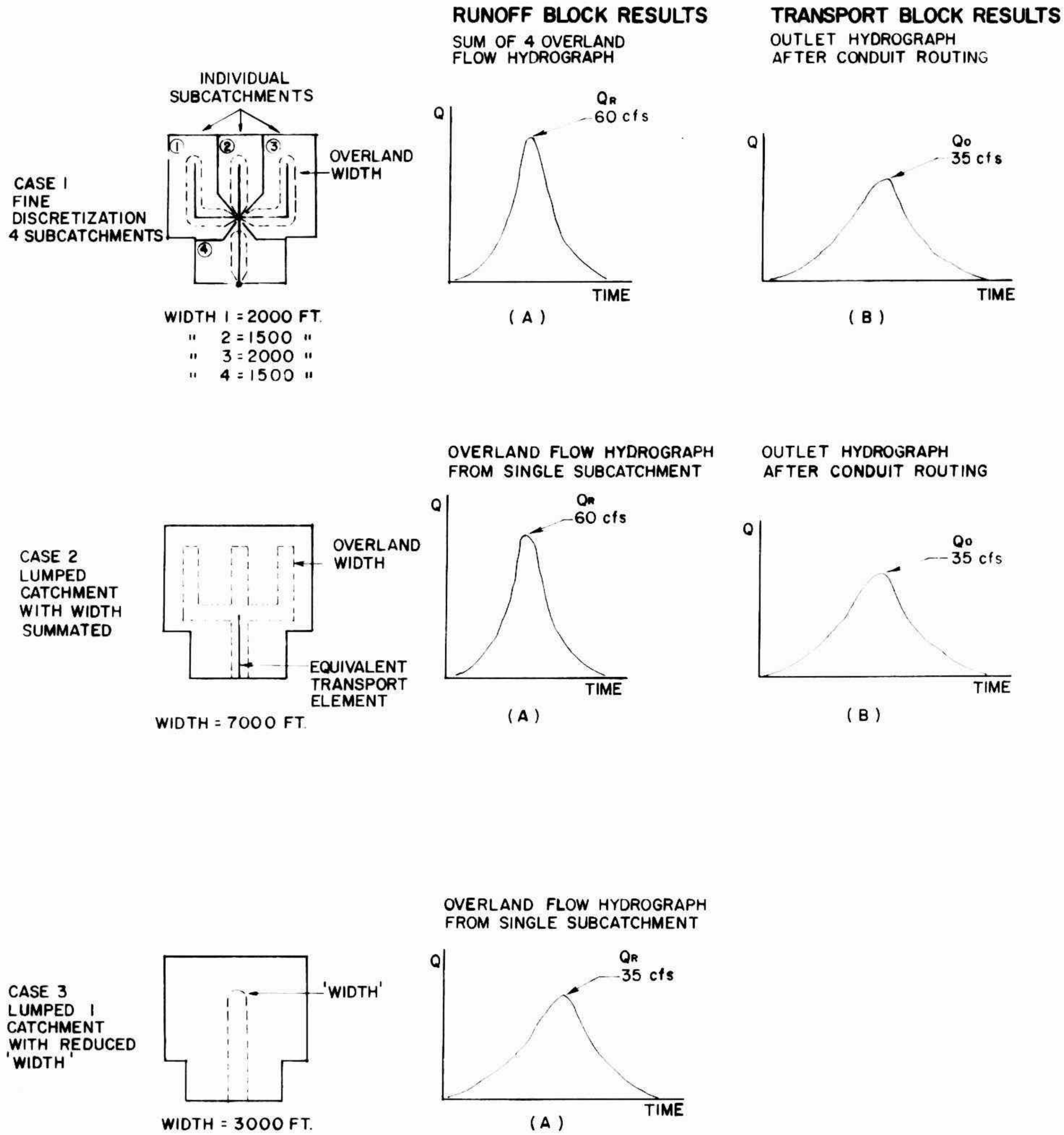
EFFECT OF VARYING OVERLAND FLOW "WIDTH" ON SURFACE RUNOFF FROM HYPOTHETICAL II ACRE AREA

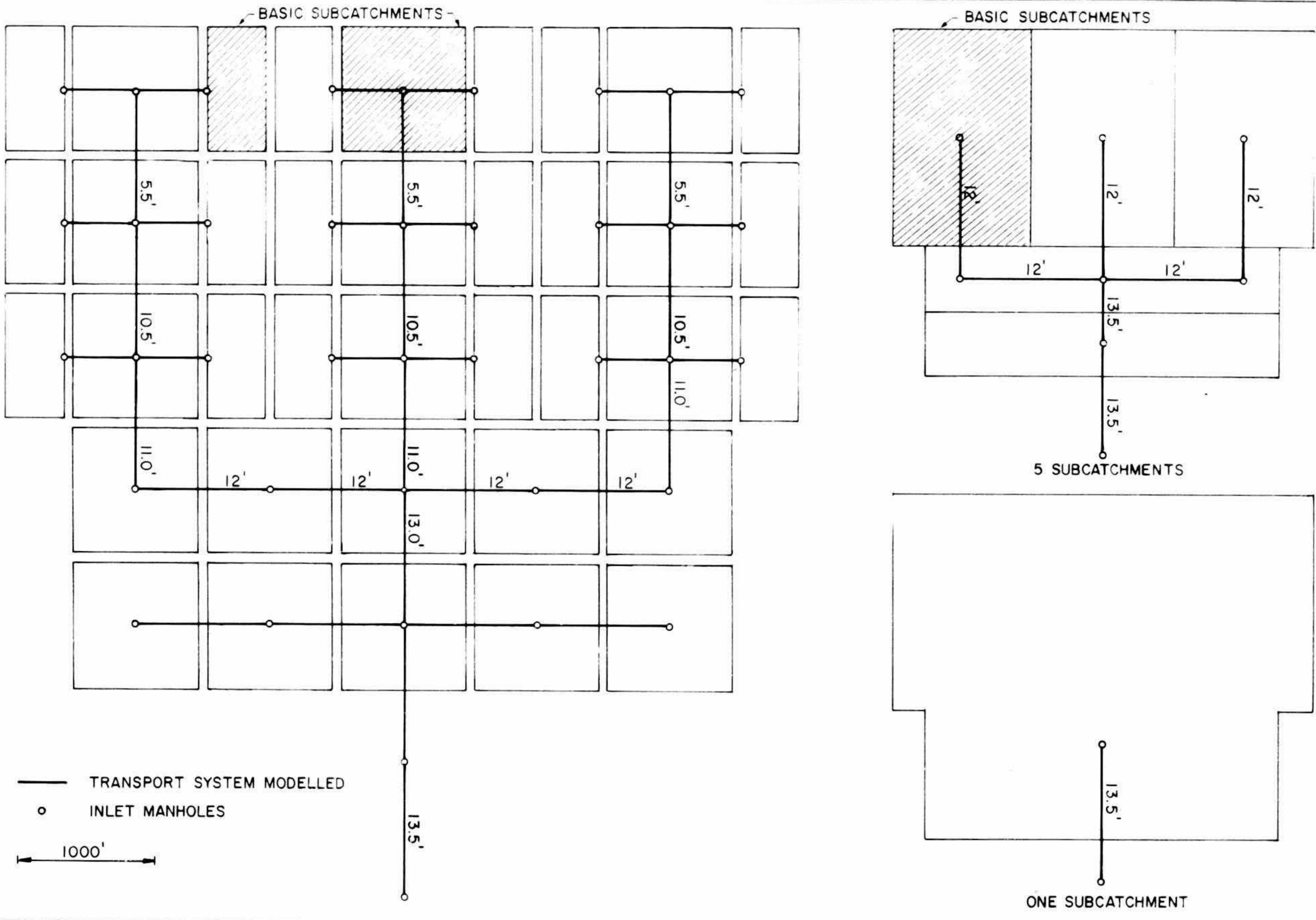
FIG. 9.2



SCHEMATIC ILLUSTRATION OF METHODS OF LUMPING

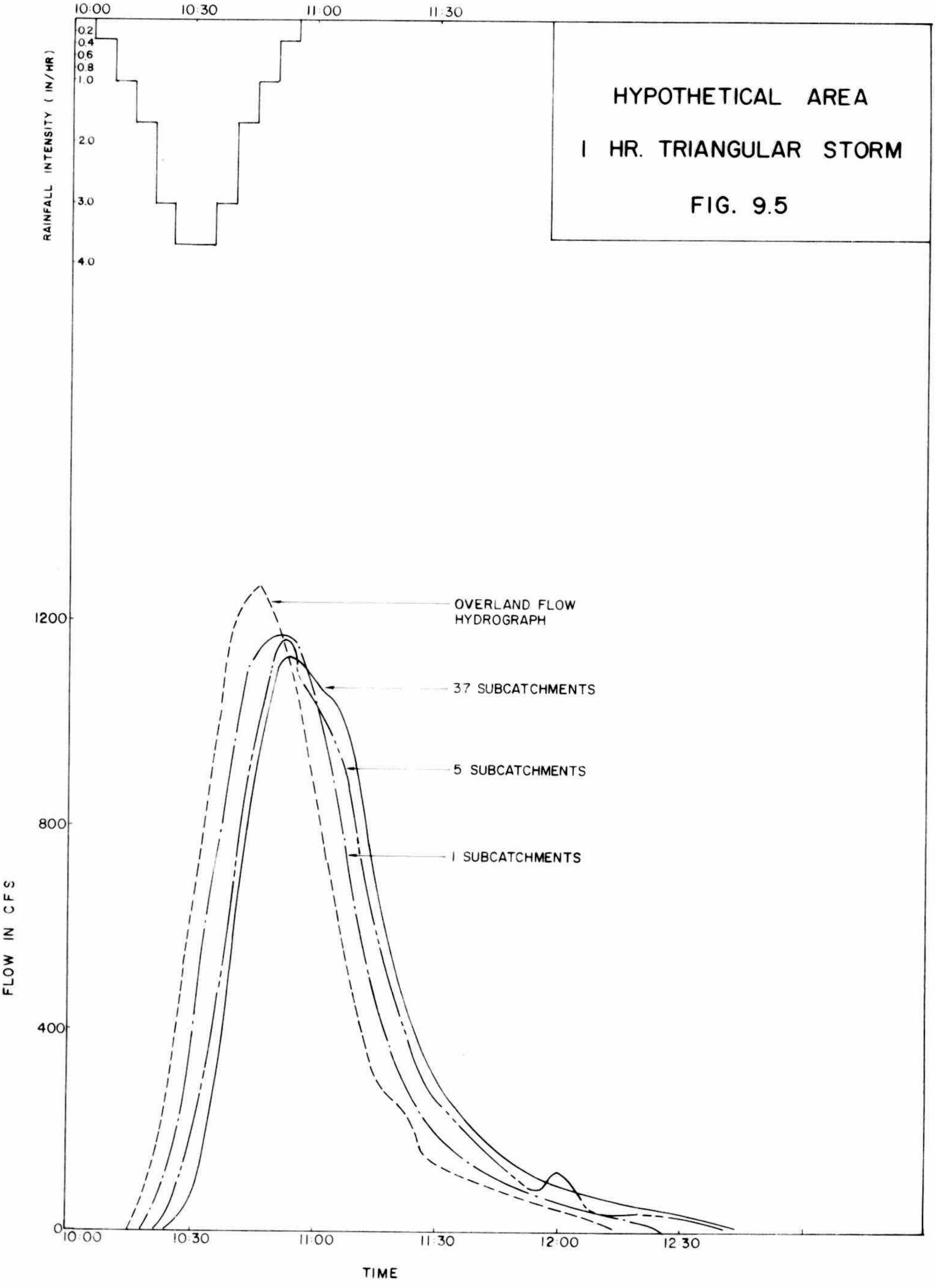
FIG. 9.3



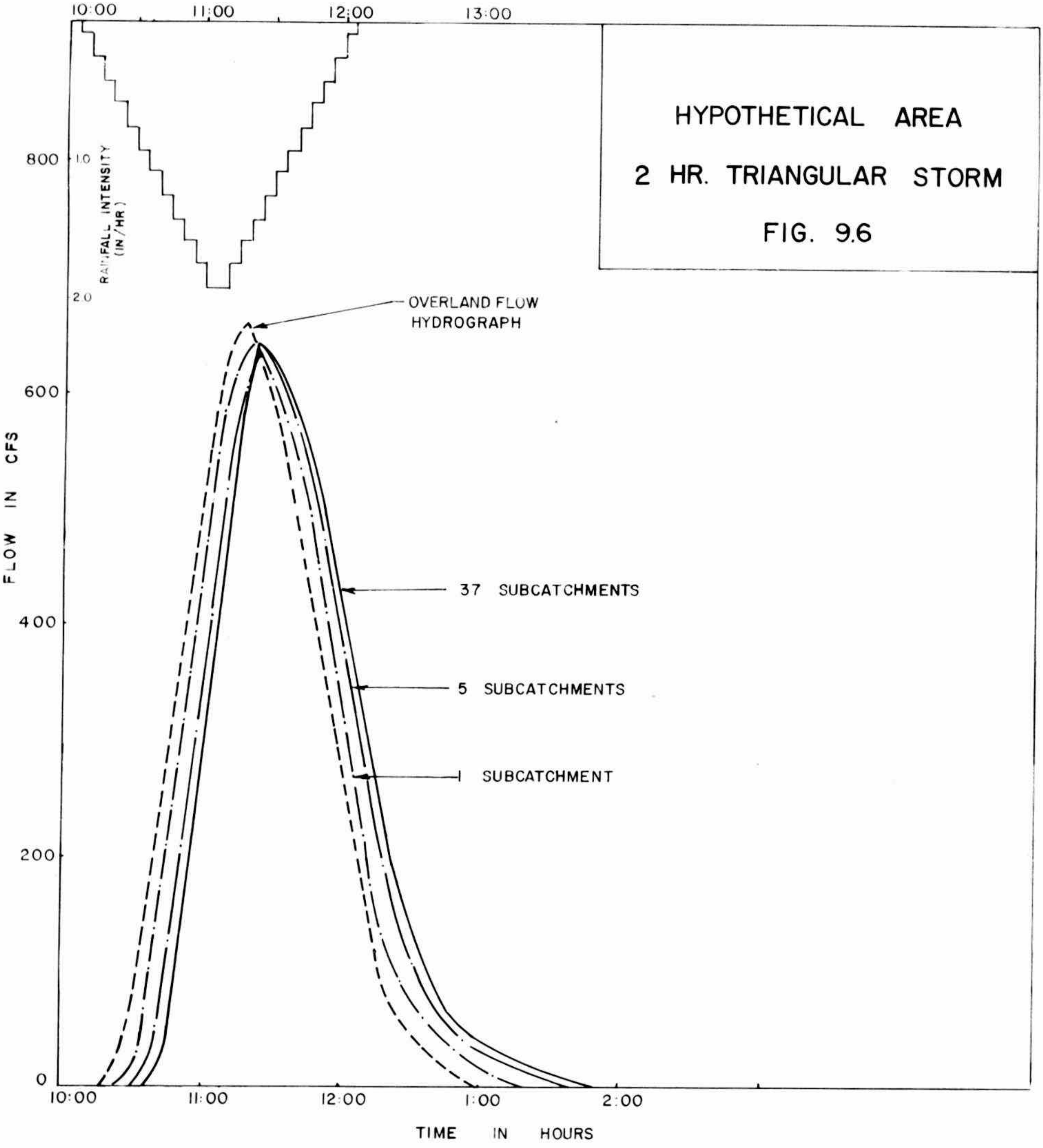


DIFFERENT LEVELS OF DISCRETIZATION OF THE HYPOTHETICAL TEST AREA

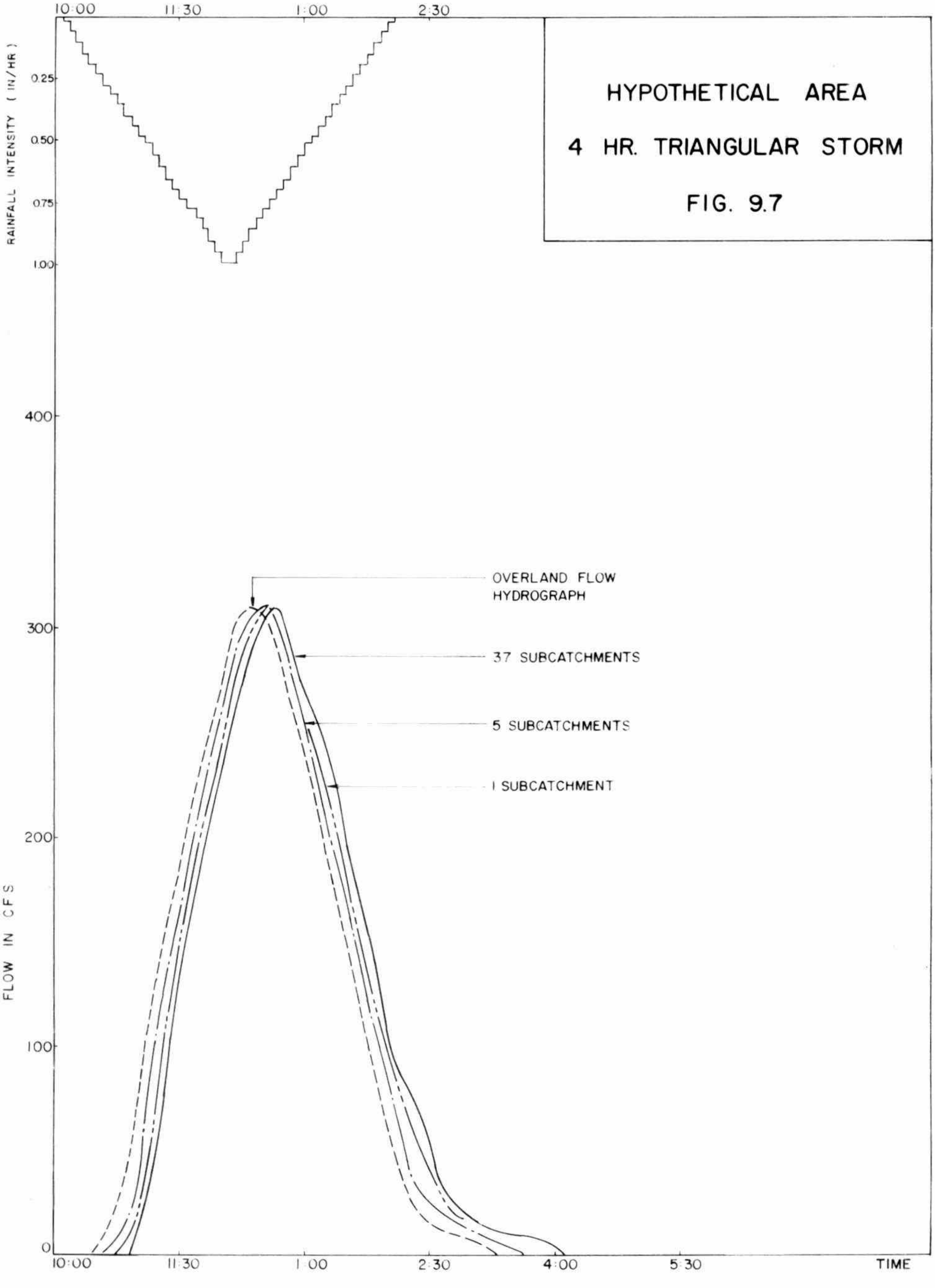
FIG. 9.4



10:00
10:30
11:00
11:30
RAINFALL INTENSITY (IN/HR)
0.2
0.4
0.6
0.8
1.0
2.0
3.0
4.0
HYPOTHETICAL AREA
1 HR. TRIANGULAR STORM
FIG. 9.5
OVERLAND FLOW HYDROGRAPH
37 SUBCATCHMENTS
5 SUBCATCHMENTS
1 SUBCATCHMENTS
1200
800
400
0
FLOW IN CFS
10:00
10:30
11:00
11:30
12:00
12:30
TIME



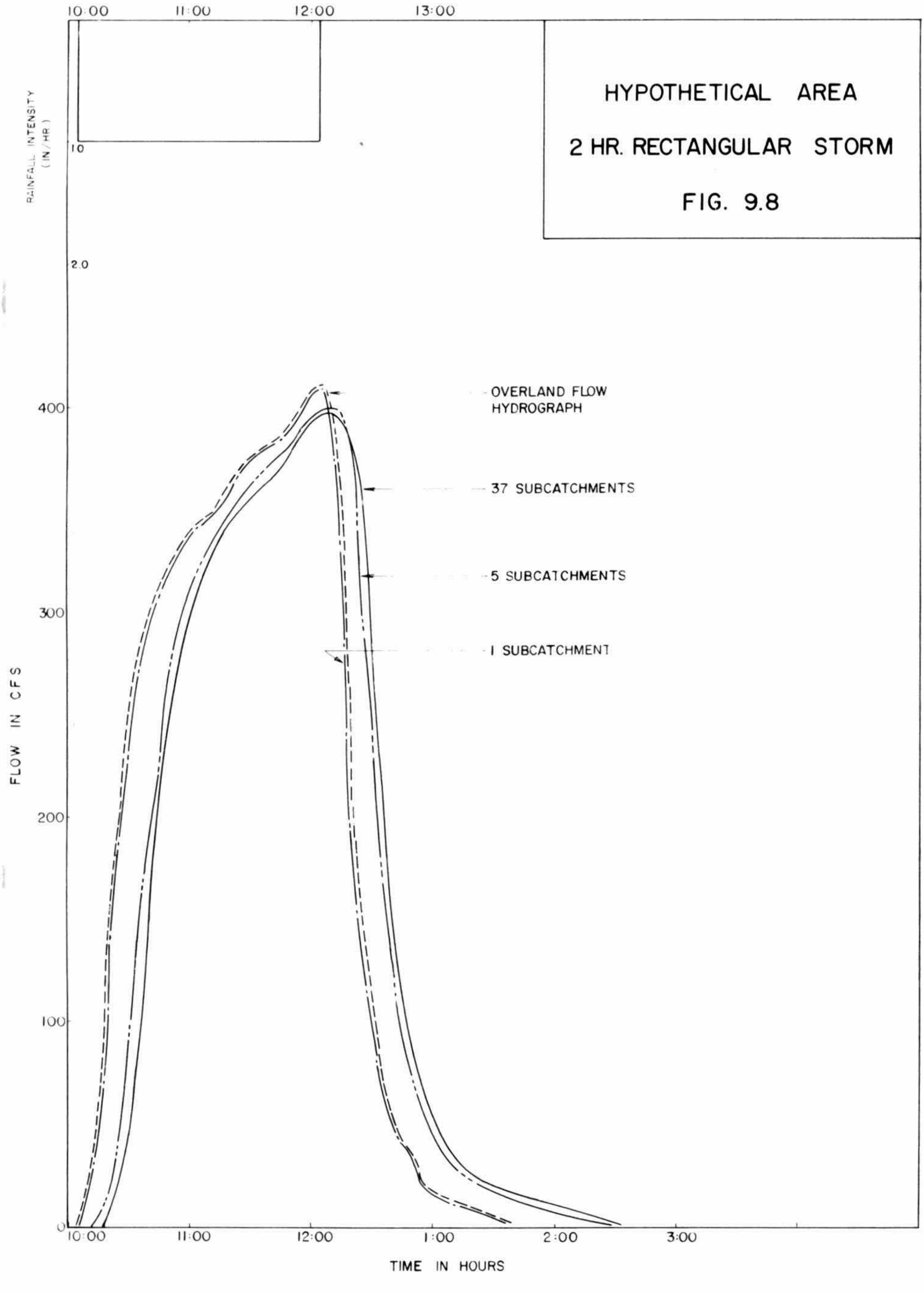
HYPOTHETICAL AREA
2 HR. TRIANGULAR STORM
FIG. 9.6
10:00
11:00
12:00
13:00
RAINFALL INTENSITY (IN/HR)
1.0
2.0
800
600
400
200
0
FLOW IN CFS
OVERLAND FLOW HYDROGRAPH
37 SUBCATCHMENTS
5 SUBCATCHMENTS
1 SUBCATCHMENT
10:00
11:00
12:00
1:00
2:00
TIME IN HOURS



HYPOTHETICAL AREA

4 HR. TRIANGULAR STORM

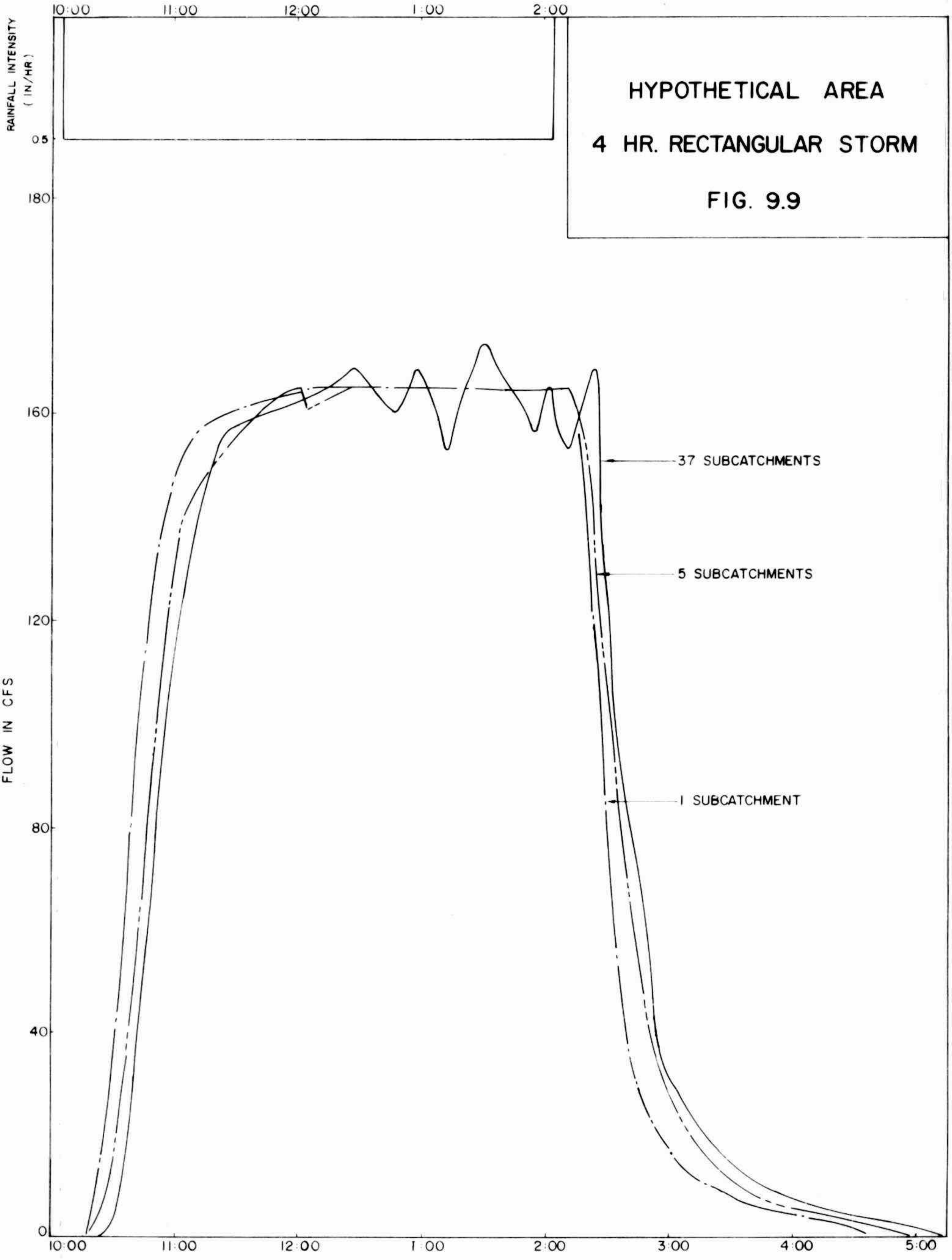
FIG. 9.7



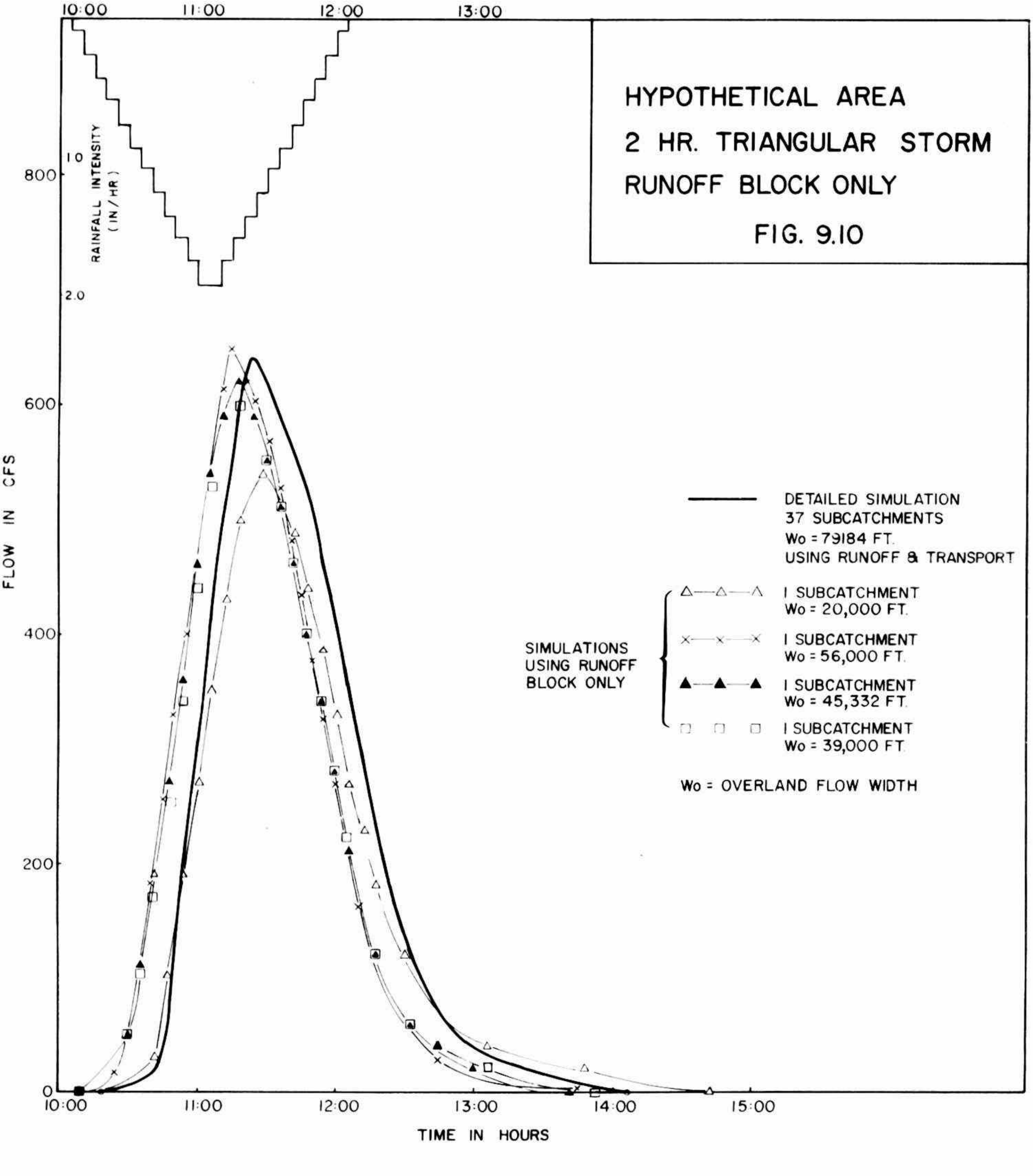
HYPOTHETICAL AREA

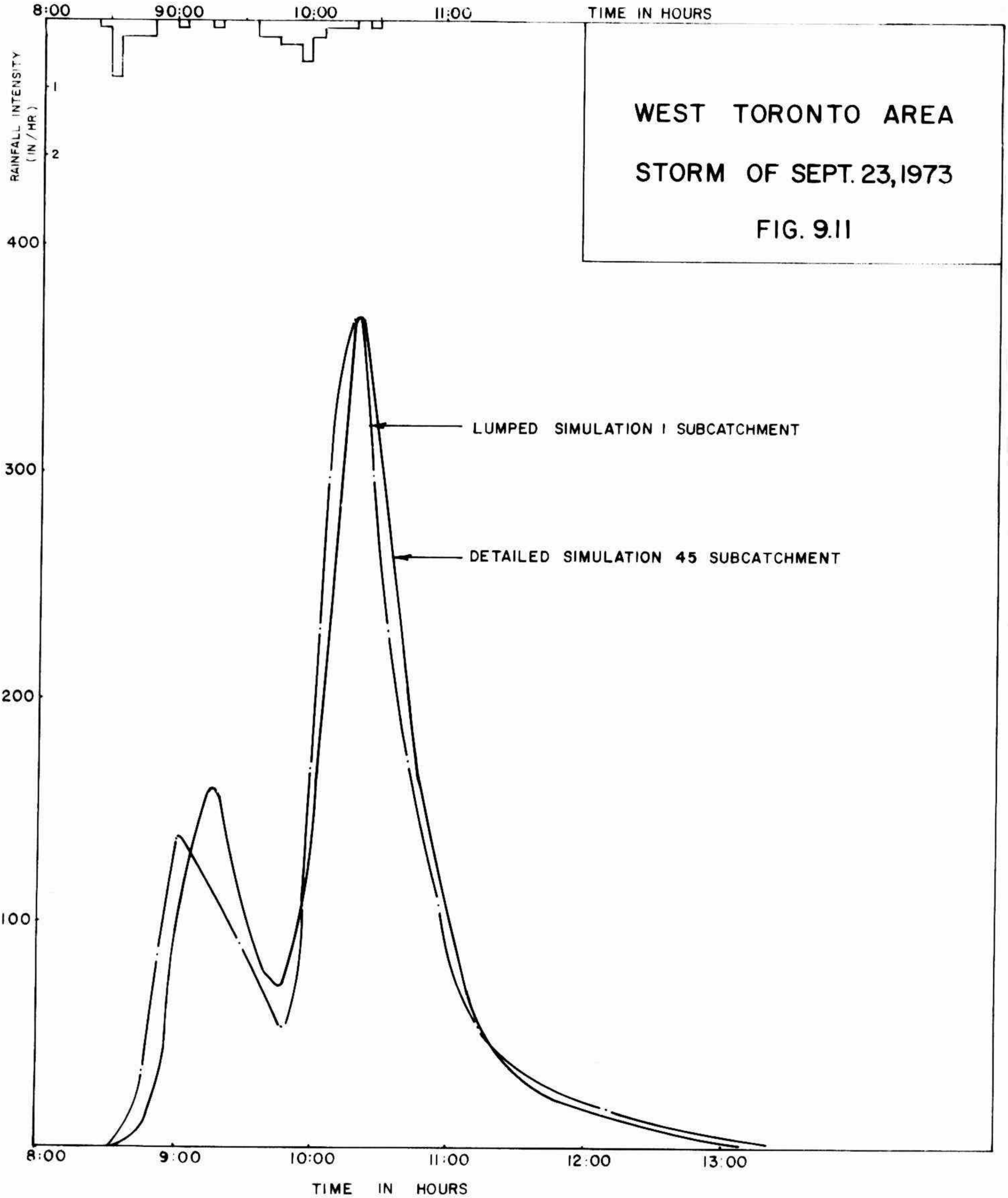
2 HR. RECTANGULAR STORM

FIG. 9.8



HYPOTHETICAL AREA
4 HR. RECTANGULAR STORM
FIG. 9.9
RAINFALL INTENSITY (IN/HR)
0.5
10:00
11:00
12:00
1:00
2:00
FLOW IN CFS
180
160
120
80
40
0
37 SUBCATCHMENTS
5 SUBCATCHMENTS
1 SUBCATCHMENT
3:00
4:00
5:00
TIME

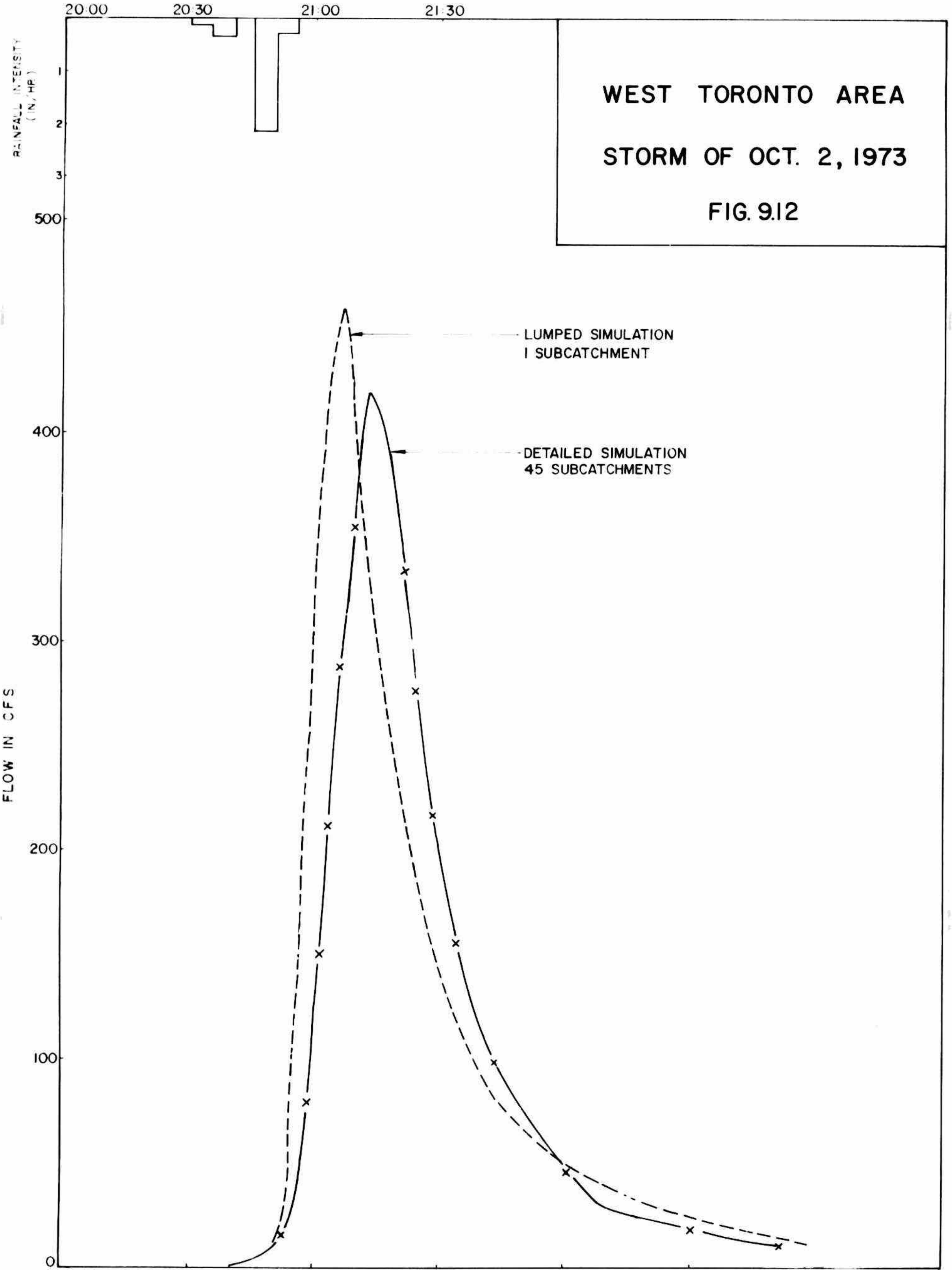

HYPOTHETICAL AREA
2 HR. TRIANGULAR STORM
RUNOFF BLOCK ONLY
FIG. 9.10
10:00
11:00
12:00
13:00
RAINFALL INTENSITY (IN / HR)
1.0
2.0
FLOW IN CFS
800
600
400
200
0
DETAILED SIMULATION
37 SUBCATCHMENTS
Wo = 79184 FT.
USING RUNOFF & TRANSPORT
SIMULATIONS USING RUNOFF BLOCK ONLY
I SUBCATCHMENT Wo = 20,000 FT.
I SUBCATCHMENT Wo = 56,000 FT.
I SUBCATCHMENT Wo = 45,332 FT.
I SUBCATCHMENT Wo = 39,000 FT.
Wo = OVERLAND FLOW WIDTH
10:00
11:00
12:00
13:00
14:00
15:00
TIME IN HOURS



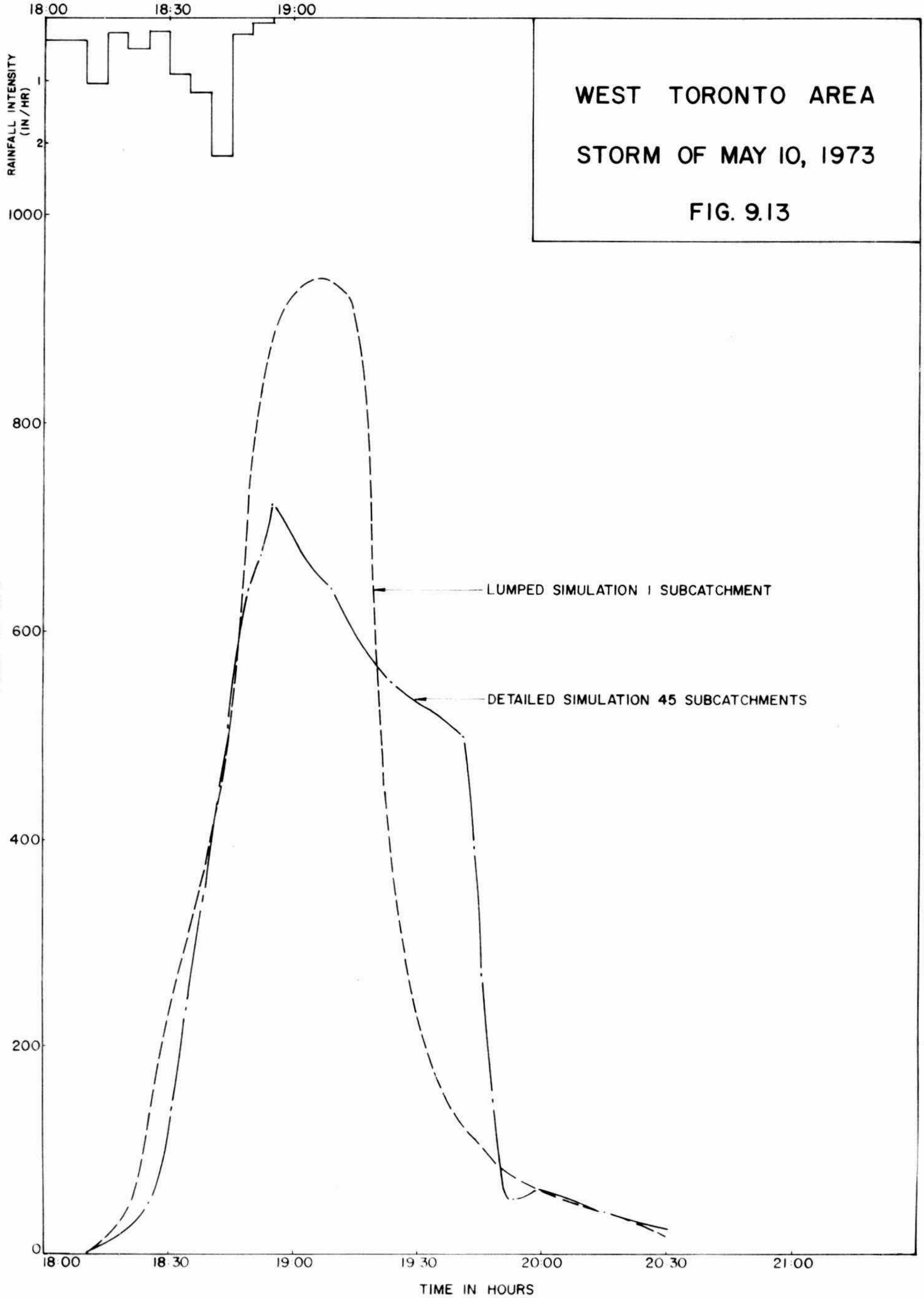
WEST TORONTO AREA

STORM OF SEPT. 23, 1973

FIG. 9.11



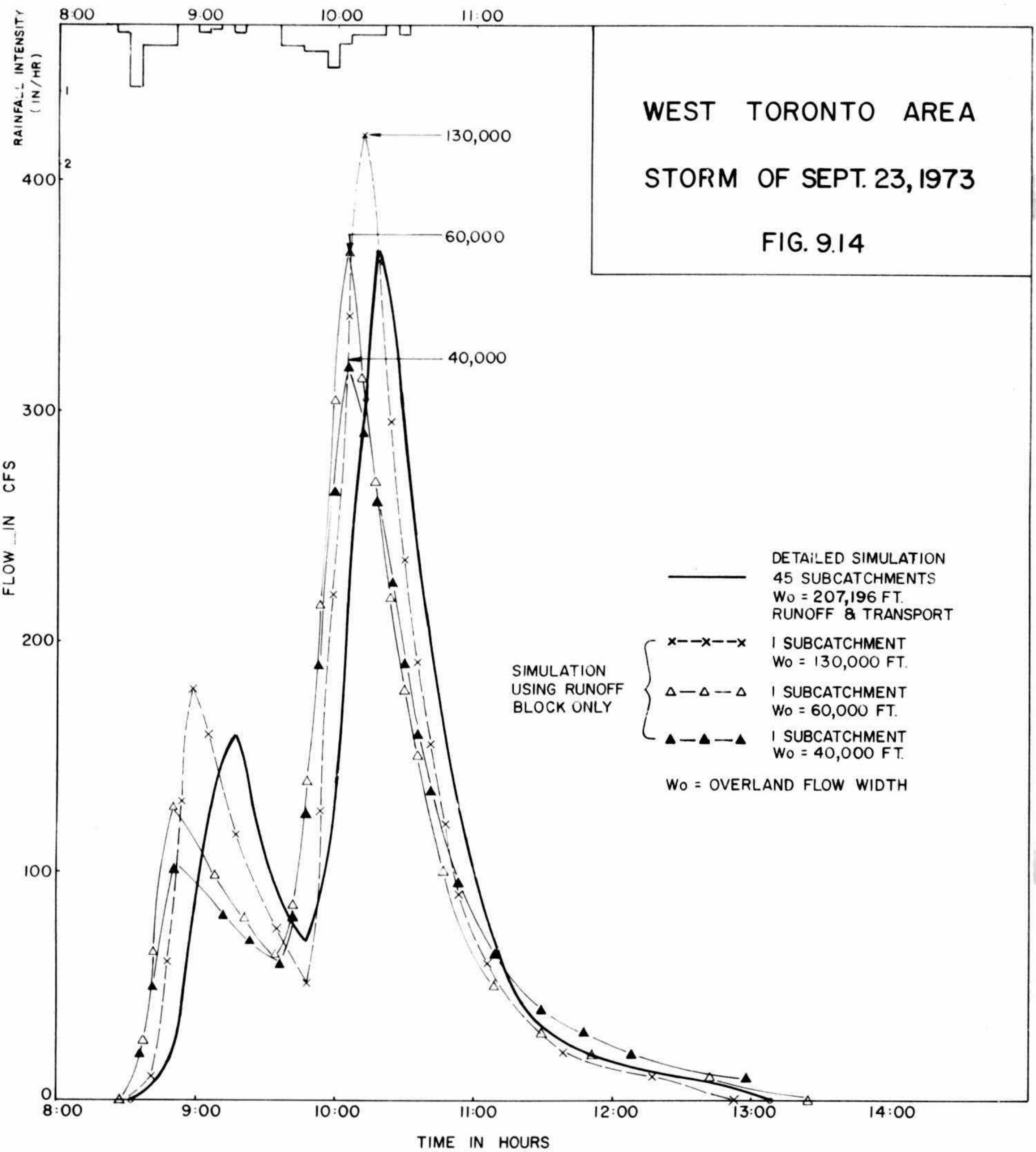
WEST TORONTO AREA
STORM OF OCT. 2, 1973
FIG. 9.12
RAINFALL INTENSITY (IN./HR.)
20:00
20:30
21:00
21:30
1
2
3
500
400
300
200
100
0
FLOW IN CFS
LUMPED SIMULATION
1 SUBCATCHMENT
DETAILED SIMULATION
45 SUBCATCHMENTS
20:00
20:30
21:00
21:30
22:00
22:30
23:00
TIME IN HOURS

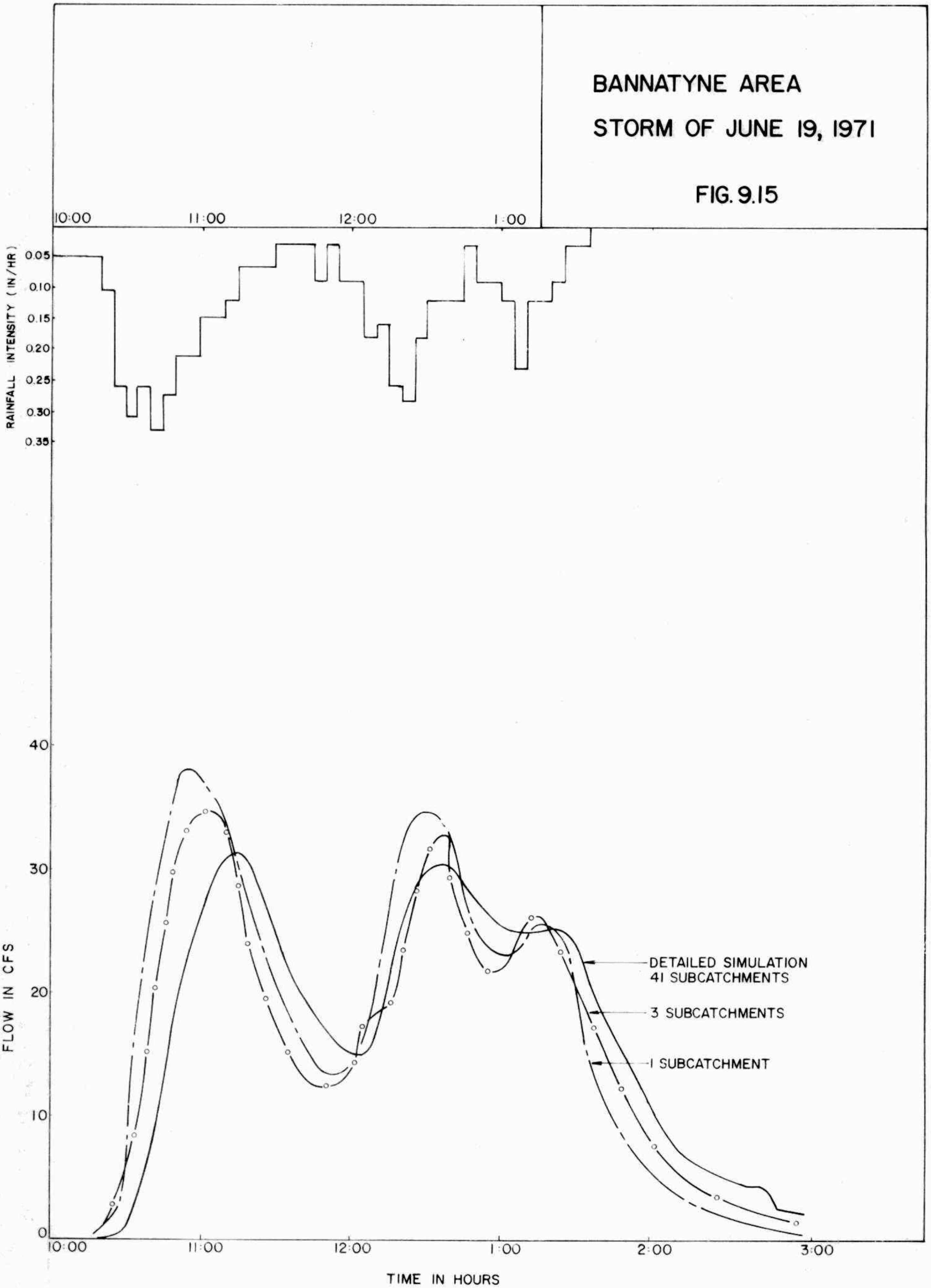


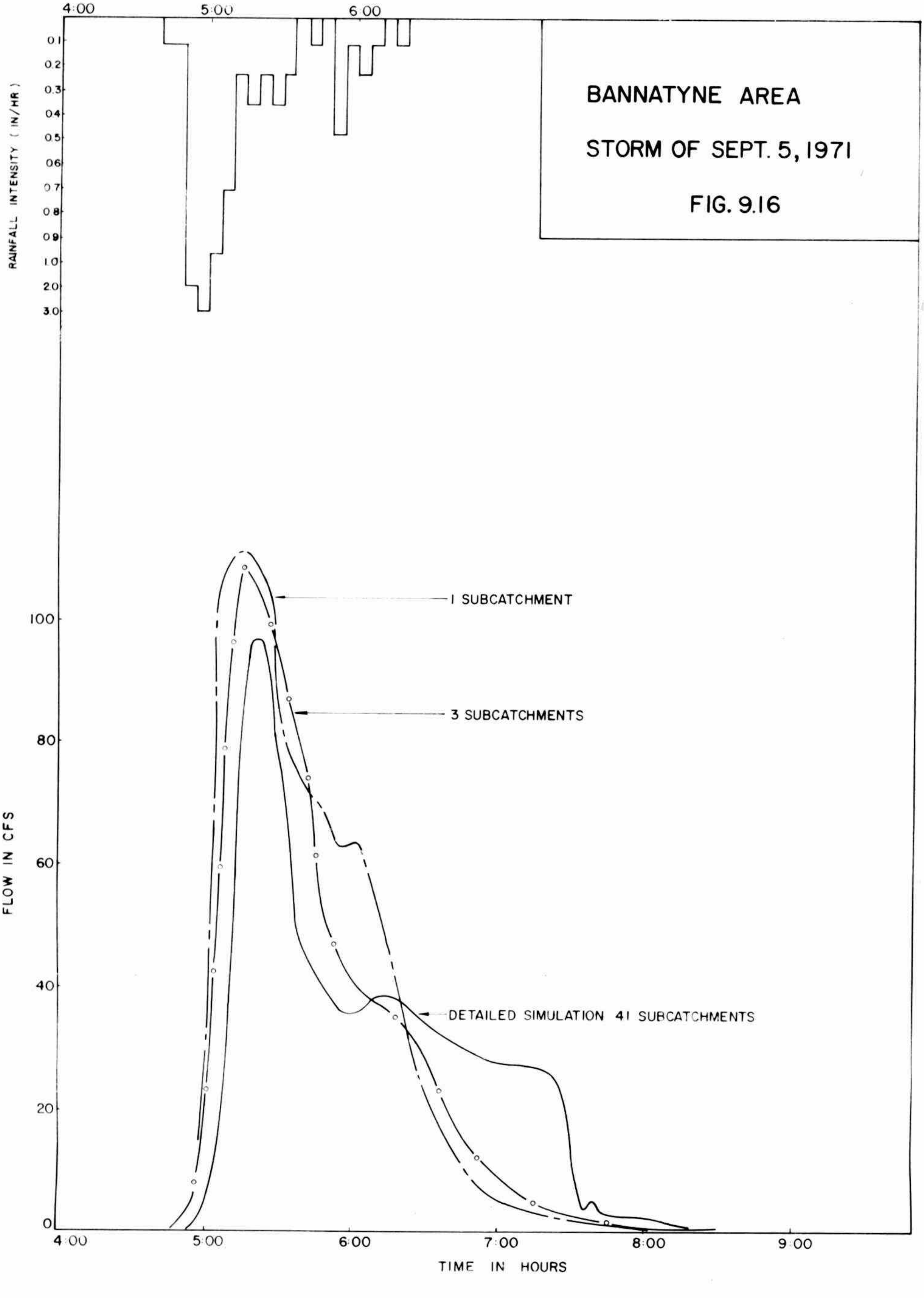
WEST TORONTO AREA

STORM OF MAY 10, 1973

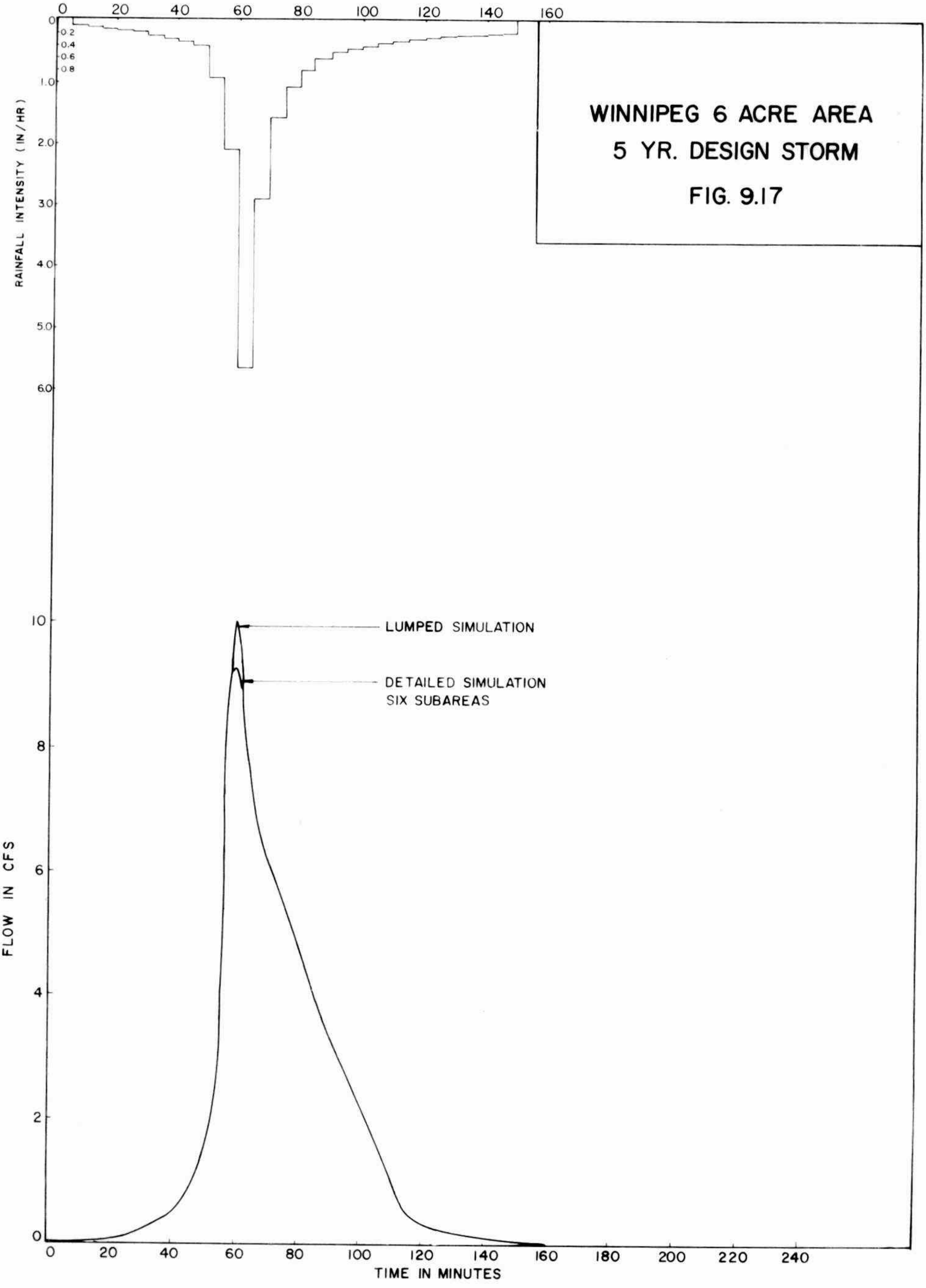
FIG. 9.13


WEST TORONTO AREA
STORM OF SEPT. 23, 1973
FIG. 9.14
RAINFALL INTENSITY (IN/HR)
FLOW IN CFS
TIME IN HOURS
130,000
60,000
40,000
DETAILED SIMULATION
45 SUBCATCHMENTS
Wo = 207,196 FT.
RUNOFF & TRANSPORT
SIMULATION USING RUNOFF BLOCK ONLY
I SUBCATCHMENT
Wo = 130,000 FT.
I SUBCATCHMENT
Wo = 60,000 FT.
I SUBCATCHMENT
Wo = 40,000 FT.
Wo = OVERLAND FLOW WIDTH


BANNATYNE AREA
STORM OF JUNE 19, 1971
FIG. 9.15
10:00
11:00
12:00
1:00
RAINFALL INTENSITY (IN/HR)
0.05
0.10
0.15
0.20
0.25
0.30
0.35
40
30
20
10
0
FLOW IN CFS
DETAILED SIMULATION 41 SUBCATCHMENTS
3 SUBCATCHMENTS
1 SUBCATCHMENT
10:00
11:00
12:00
1:00
2:00
3:00
TIME IN HOURS



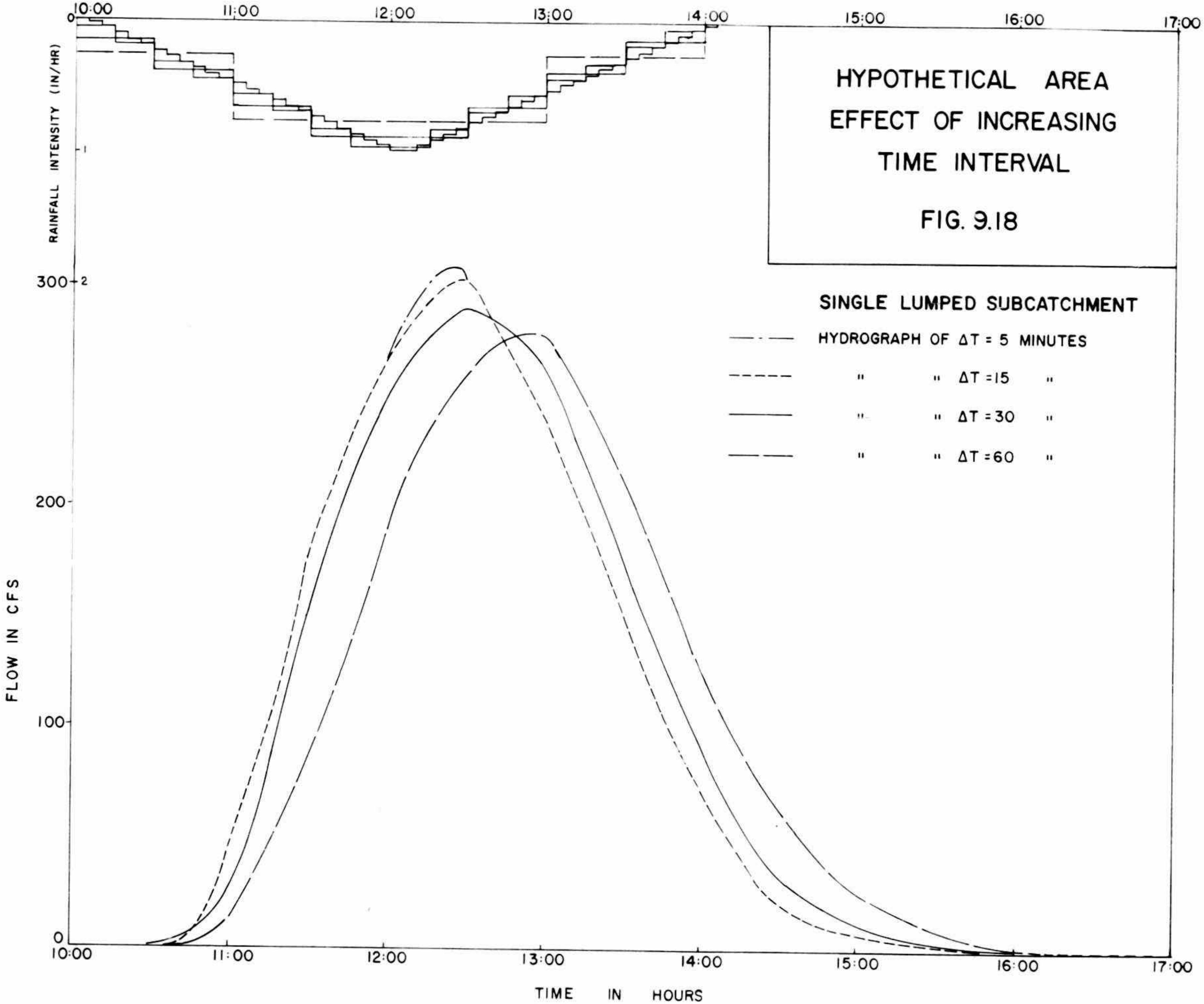
BANNATYNE AREA
STORM OF SEPT. 5, 1971
FIG. 9.16
RAINFALL INTENSITY (IN/HR)
4:00
5:00
6:00
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2.0
3.0
1 SUBCATCHMENT
3 SUBCATCHMENTS
DETAILED SIMULATION 41 SUBCATCHMENTS
FLOW IN CFS
100
80
60
40
20
0
4:00
5:00
6:00
7:00
8:00
9:00
TIME IN HOURS



WINNIPEG 6 ACRE AREA
5 YR. DESIGN STORM
FIG. 9.17

HYPOTHETICAL AREA
EFFECT OF INCREASING TIME INTERVAL

FIG. 9.18



A GENERALIZED SWMM QUALITY MODEL

CHAPTER 10

CHAPTER 10
A GENERALIZED SWMM QUALITY MODEL

10.1 GENERAL

As indicated in Chapter 5, the current state of the art of stormwater quality modelling rarely justifies complex discretization of the catchment or the consideration of in-system storage. It was shown in Chapter 9 that it is possible to combine several small subcatchments into a single equivalent catchment without loss of accuracy in stormwater quantity and quality simulation. A "lumped" version of SWMM, allowing aggregation of subcatchments and variable pollutant accumulation control parameters, also shows promise as a intermediate tool for multi-event simulation.

As a next step toward simplified quality modelling, a new computer model was developed, using the basic concepts of the quality algorithm for the SWMM, in a less sophisticated formulation. An attempt was made to overcome several disadvantages of the SWMM quality routines as compared for example with those used in STORM. (This model is described in Chapter 11) In SWMM the surface pollutant loading rates are built into the program, whereas in STORM different loading rates can be read into the model for various land uses. STORM also performs quality computations based on a specified input hydrograph, if required. This permits a comparison of the pollutographs obtained from measured hydrographs or based on hydrographs from any runoff simulation model.

The Generalized Quality Model represents a practical alternative to the use of SWMM with a coarse discretization. The Generalized Model is easier to calibrate than SWMM because all parameters may be readily adjusted. The new model is potentially more accurate for the modelling of pollutant washoff than STORM [1] because of the shorter time intervals used for washoff computations. The model was tested using data from the Brucewood area.

10.2 DISCUSSION OF BASIC CONCEPTS

The pollutant washoff rate is computed from the exponential decay equation which is used in both SWMM and STORM. Two different methods for the computation of suspended solids are included in the Generalized Quality Model. The first (ISS=0) uses the exponential washoff equation and allows for a variable availability factor. The second (ISS=1) is based on an empirical equation. (Both equations are given in Figure 5-1).

In the Generalized Quality Model the amounts of surface pollutants available from each of the five possible land use areas are established from a daily dust and dirt loading rate for each land use classification. The proportions of each pollutant in the accumulated dust and dirt are specified by the user. The daily loading rate is multiplied by the number of dry days and the total surface pollutant load is the sum of the contributions from all the land uses. This total amount can be modified according to the frequency and efficiency of street sweeping as in the SWMM. This approach is equivalent to estimating a weighted average pollutant loading for a single equivalent catchment.

In the case that no local data describing dust and dirt loading rates, and pollutant composition of dust and dirt are available, the values given in the APWA survey may be used as a first estimate [2].

10.3 DESCRIPTION OF THE GENERALIZED QUALITY MODEL

The Generalized Quality Model computes surface runoff quality in basically the same manner as the SWMM. Some of the important aspects of the new model are summarized below:

(a) Aggregated single catchment Model

(b) Five land uses possible

(c) Separate input hydrograph forms basis for quality computations

(d) User supplied dust and dirt composition and loading rates for each land use

(e) No quality routing or pollutant decay computations

(f) No erosion or deposition of sediments computed

(g) Two methods for Suspended Solids computation

(h) Catchbasin contributions modelled

(i) Quality calculations may be based on either flow from the total area, or only on flow from the impervious area

(j) Ten pollutants may be simulated (BOD, COD, Suspended Solids, Settleable Solids, Coliforms, N, PO_4, Cl, lead, oil and grease)

(k) No default values supplied

(l) Pollutographs, mass curves and surface load statistics are available for each pollutant

The principal advantage of the Generalized Quality Model is the reduced cost of computations and reduced data preparation time achieved by aggregating the properties of the entire study area and modelling it as a single catchment. The model is extremely flexible and may be used for the study of many aspects of stormwater pollution.

Five different land uses are allowed in the single aggregated catchment:

Single family residential
Multiple family residential
Commercial
Industrial
Open Space

Dust and dirt loading rates and compositions are required for each land use and the total accumulated amounts are computed according to the total area in each category. Generally only regional average pollutant loading rates will be available. However the user can readily adjust these for calibration purposes. The model does not supply default values for parameters such as street sweeping frequency and efficiency, number of catchbasins, number of dry days preceding the storm event and pollutant availability and washoff coefficient. Consequently, known values or calibrated estimates may be readily supplied and the model can be quickly adjusted for local conditions.

The overland flow hydrograph has to be supplied to the Generalized Quality Model. This means that this hydrograph may be generated using the SWMM or any other urban runoff model, or measured flows can be used.

Two flow options have been included in the model. In the first, the washoff intensity is computed assuming the hydrograph is generated only from impervious areas. This approach is used in STORM for quality calculations and would normally be used for low intensity storms, where infiltration on the pervious areas predominates and results in little or no runoff from these areas. The second method, which would normally be used for higher intensity storms or in areas with low infiltration rates, assumes the hydrograph to have been generated over the entire area.

The model does not consider pipe flow or surface erosion principally in order to minimize time required for data preparation and computational costs.

10.4 VERIFICATION OF THE GENERALIZED QUALITY MODEL

Preliminary calibration and verification of the Generalized Quality Model was carried out using data from the Brucewood and Barrington study areas. The detailed SWMM simulations for Brucewood are discussed in Chapter 5. The Barrington catchment in East York is served by a separate sewer system draining an area of about 56 acres. The principal land use is single family residential.

The results presented in this section demonstrate the flexibility of the model and its application in assessing the sensitivity to various parameters in order to establish a well calibrated model. The main parameters used for calibration were:

(a) The availability factor for Suspended Solids washoff

(b) The removing coefficient CC

(c) The exponent b

The measured dust and dirt accumulation rates and the available values for catchbasin BOD were used for the Brucewood simulations. Otherwise the SWMM default values were used.

The recorded storms are of relatively low intensity. In some cases the flows generated from previous detailed SWMM simulations were used as input to the model in order to compare the results based on these with those based on simulations using the measured flows.

Figures 10-1 to 10-5 show the results of the simulations of the Brucewood flows. An additional Figure, 10-6, shows a simulation for the Barrington catchment.

(a) Figure 10-1

The empirical equation (ISS=1) was used for suspended solids computations. The pollutographs were generated based on both the measured and the SWMM hydrographs. There was only a marginal difference between the simulations resulting from use of these alternative flows. However, a significant improvement is evidently caused by a reduction in the removing coefficient, CC, from 0.9 to 0.25.

(b) Figure 10-2

The best fit to the measured pollutographs was obtained using the empirical equation for suspended solids computations (ISS=1) with a removing coefficient, CC, of 0.25. The equivalent SWMM results are shown for comparison. The accuracy of the Generalized Model simulation was not significantly altered by use of the measured flows as opposed to the SWMM hydrographs. A removing coefficient of 0.9 resulted in large overestimation of the peak concentrations, while use of the exponential equation for suspended solids (ISS=0) failed to reproduce the shape of the measured pollutographs. This is probably due to the low initial surface loads combined with low rainfall intensities.

(c) Figure 10-3

The best simulations of the recorded pollutographs for both BOD and suspended solids were obtained using a removing coefficient of 0.5 with the empirical equation (ISS=1) for suspended solids. It was found that the use of the recorded flows instead of the SWMM hydrograph did not significantly affect the accuracy of the simulation. The equivalent SWMM results are shown for comparison.

(d) Figure 10-4

In this simulation the measured dust and dirt accumulation rate, during the dry period prior to the event, of 0.5 lbs./day/100 feet of curb was supplied to the model. In addition, the suspended solids composition of the dust and dirt was set at 830 mg of suspended solids per gm of dust and dirt, while the proportion of BOD remained the same as the SWMM default value. Good results for suspended solids and reasonable accuracy for BOD are obtained using a removing coefficient CC = 0.5.

Once again, use of the exponential suspended solids equation lead to a distortion of the pollutograph, emphasizing that for low intensity storms, even with an availability factor of 1.0 for suspended solids, this equation is not applicable. The results of the equivalent SWMM simulation are shown for comparison.

(e) Figure 10-5

In this case the measured dust and dirt accumulation rate prior to the storm was 0.6 lbs./day/100 feet of curb (c.f. 0.7 in SWMM) with the proportion of suspended solids being 570 mg/gm. Computations were based on the flows resulting from a detailed SWMM simulation. The best results were computed using the empirical equation (ISS=1) with a removing coefficient CC = 0.5. However the suspended solids pollutograph is not very well represented.

(f) Figure 10-6

These plots summarize the results of simulations for a single storm event recorded at the Barrington catchment. The low intensity rainfall and low runoff peak are typical of the events recorded for this area. The Figure indicates the results of computations based on the empirical equation (ISS=1) with CC = 0.5, since the low initial surface pollutant loads lead to a significant underestimation when using the other option (ISS=0). The peak suspended solids concentrations computed based on both the SWMM hydrograph and the recorded flows were higher than those measured, while both methods underestimated BOD levels.

10.5 CONCLUSIONS

(a) A generalized surface runoff quality model has been developed based on the mathematical relationships and computational options currently employed in the SWMM and STORM models.

(b) The Generalized Quality Model is more flexible than the SWMM quality model as all parameters may be adjusted without program modification. This facilitates calibration and leads to slightly better simulations of surface runoff quality than those obtained using SWMM.

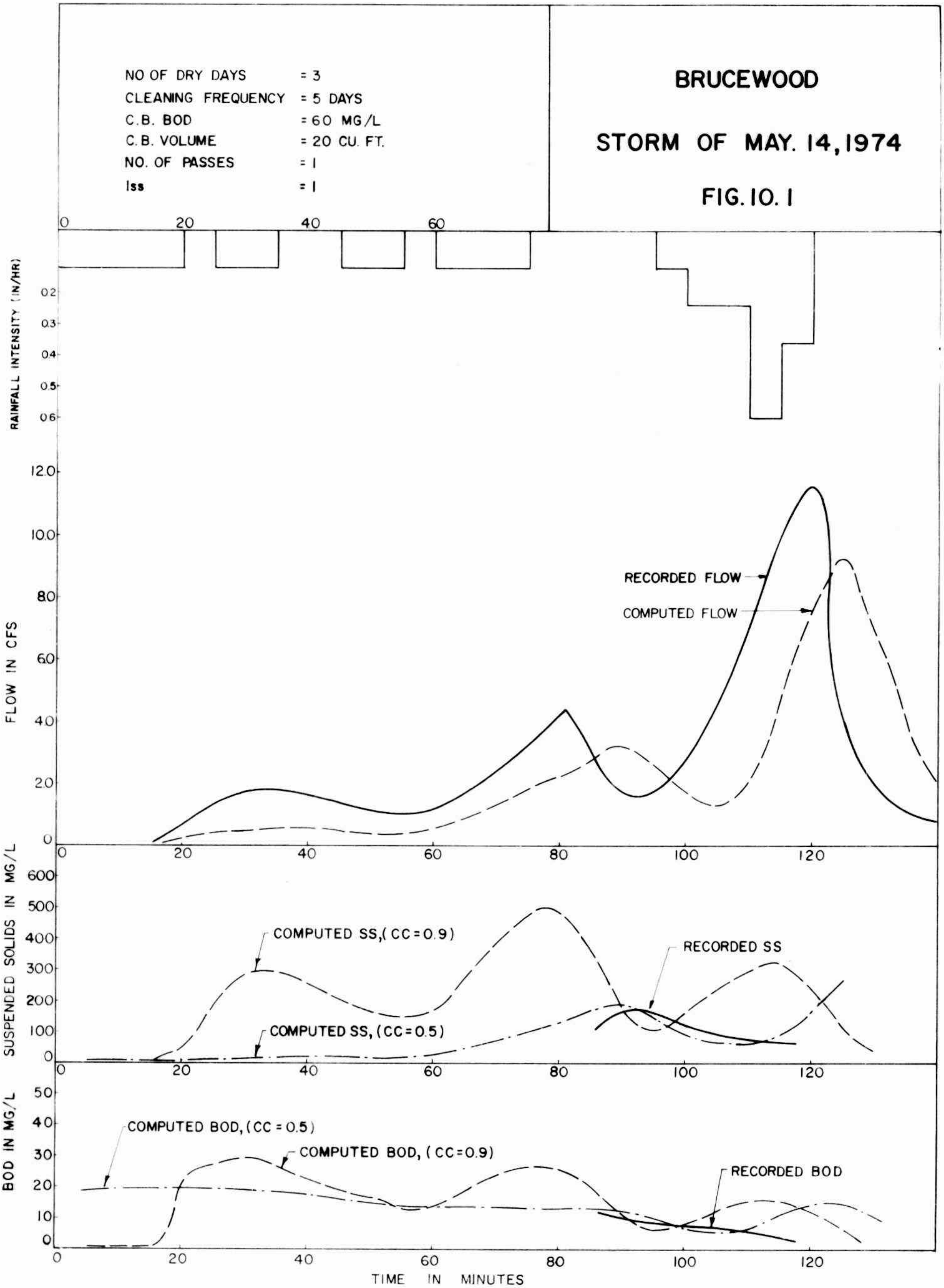
(c) The model can operate with any input hydrograph. This work has shown there is little difference between the results of the Generalized Quality Model based on the recorded flows or SWMM hydrograph.

(d) The shape of the pollutograph is generally better estimated using the empirical equation (ISS=1), although it appears that superior results may be obtained by relating the value of the removing coefficient, CC, to runoff intensity. A value of CC of 0.25 to 0.5 gave better simulations than those based on the SWMM default value of 0.9 for the low intensity storms modelled.

REFERENCE - CHAPTER 10

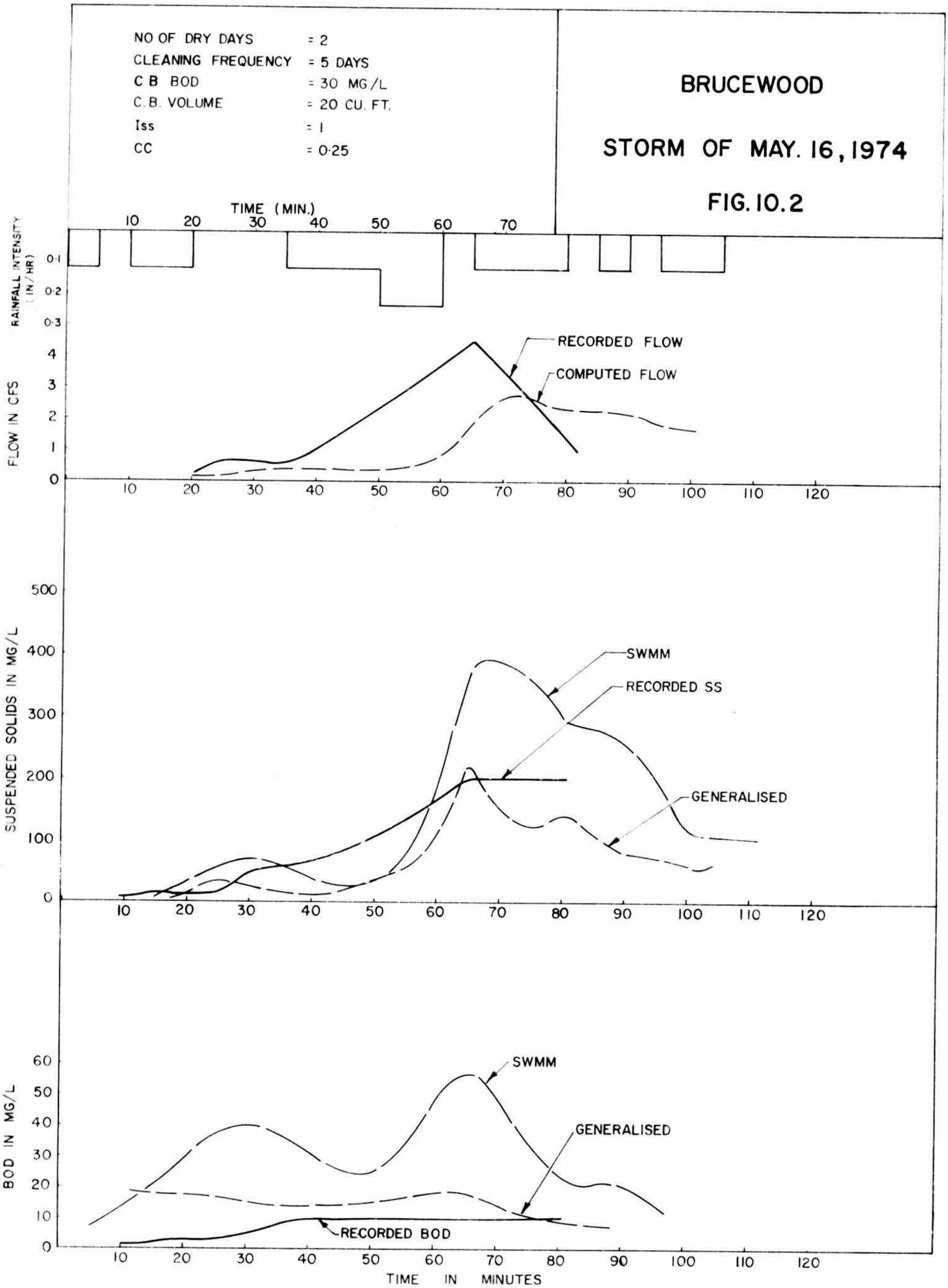
1. Urban Runoff: Storage, Treatment and Overflow Model - STORM, Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, California, September, 1973.

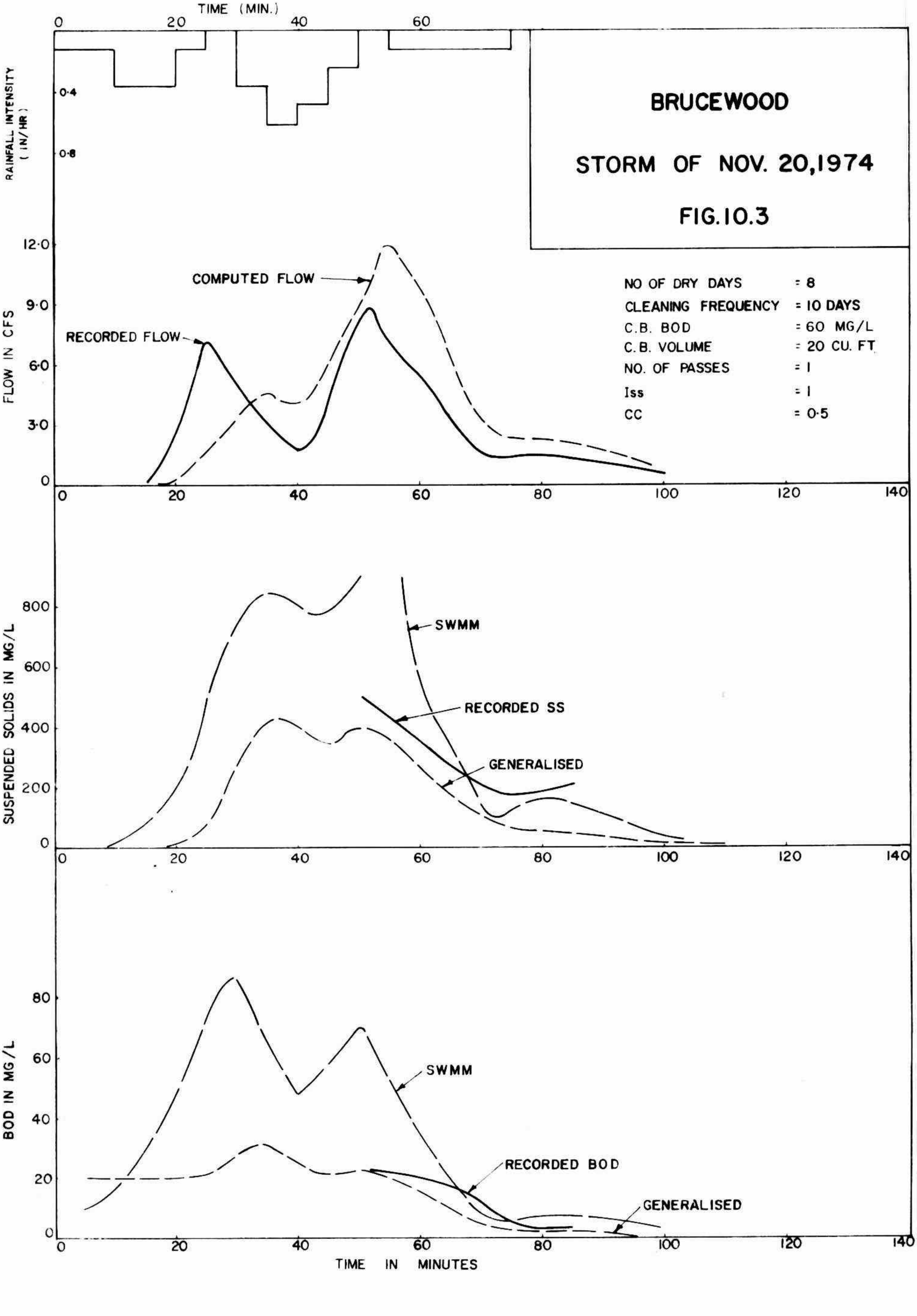
2. "Water Pollution Aspects of Urban Runoff", U.S. Department of the Interior, Federal Water Pollution Control Administration, WP-20-15, January 1969.



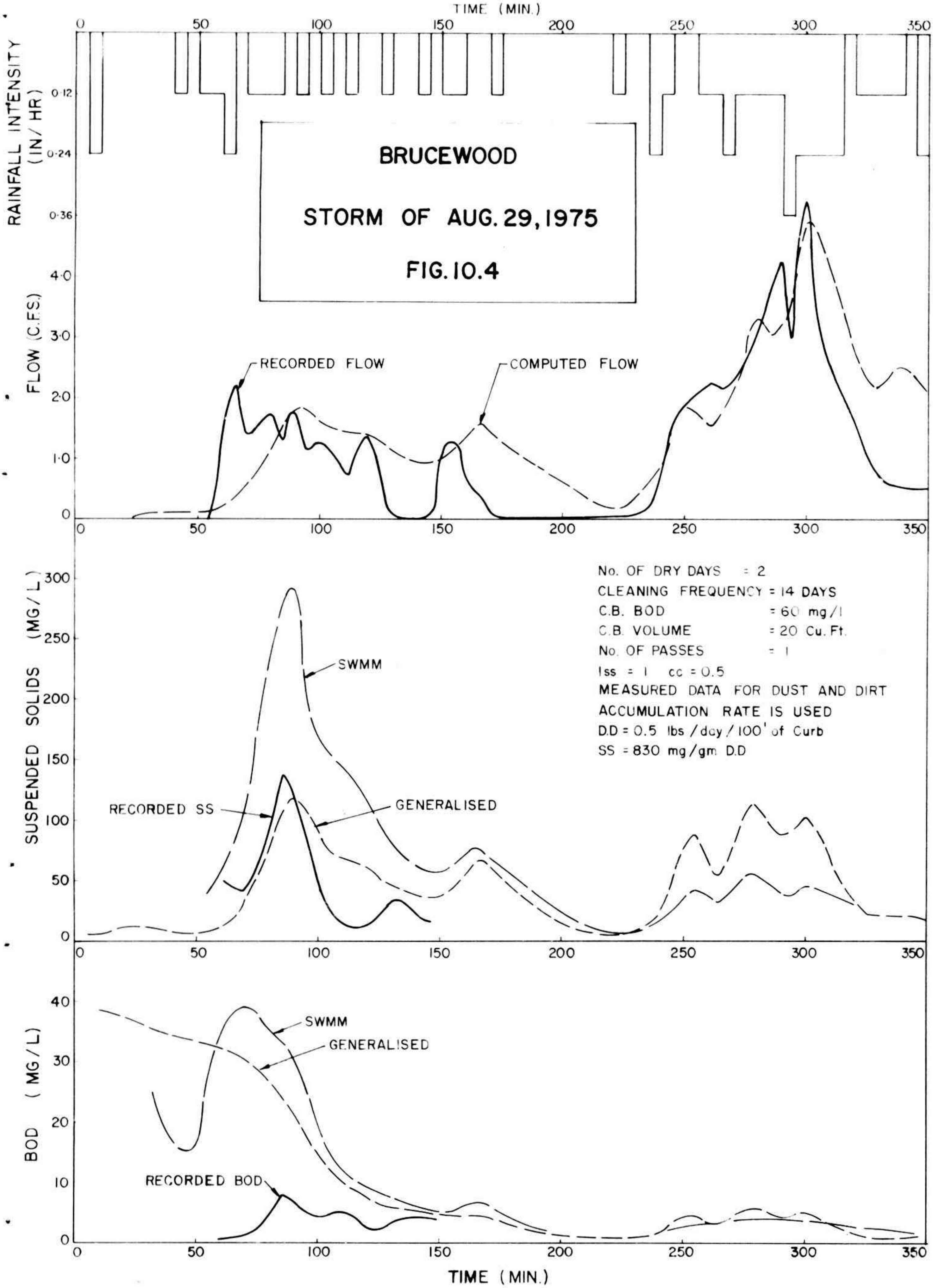
NO OF DRY DAYS = 3
CLEANING FREQUENCY = 5 DAYS
C.B. BOD = 60 MG/L
C.B. VOLUME = 20 CU. FT.
NO. OF PASSES = 1
Iss = 1
BRUCEWOOD
STORM OF MAY. 14, 1974
RAINFALL INTENSITY (IN/HR)
0.2
0.3
0.4
0.5
0.6
FLOW IN CFS
12.0
10.0
8.0
6.0
4.0
2.0
0
RECORDED FLOW
COMPUTED FLOW
SUSPENDED SOLIDS IN MG/L
600
500
400
300
200
100
COMPUTED SS, (CC = 0.9)
RECORDED SS
COMPUTED SS, (CC = 0.5)
BOD IN MG/L
50
40
30
20
10
COMPUTED BOD, (CC = 0.5)
COMPUTED BOD, (CC = 0.9)
RECORDED BOD
0
20
40
60
80
100
120
TIME IN MINUTES

FIG. 10.1

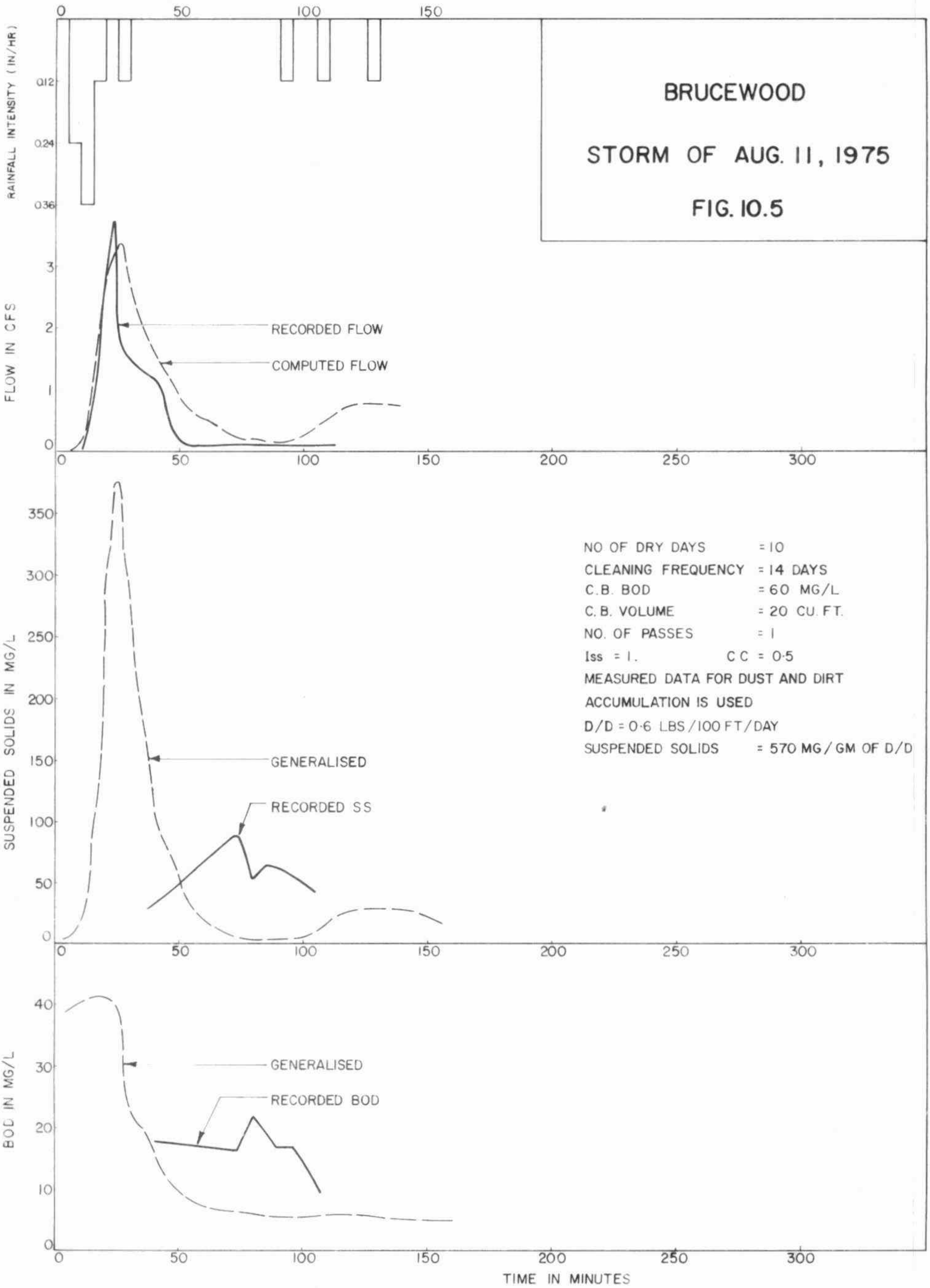

NO OF DRY DAYS = 2
CLEANING FREQUENCY = 5 DAYS
C B BOD = 30 MG/L
C.B. VOLUME = 20 CU. FT.
Iss = 1
CC = 0·25
BRUCEWOOD
STORM OF MAY. 16, 1974
FIG. 10.2
TIME (MIN.)
RAINFALL INTENSITY (IN/HR)
RECORDED FLOW
COMPUTED FLOW
FLOW IN CFS
SUSPENDED SOLIDS IN MG/L
SWMM
RECORDED SS
GENERALISED
BOD IN MG/L
SWMM
GENERALISED
RECORDED BOD
TIME IN MINUTES

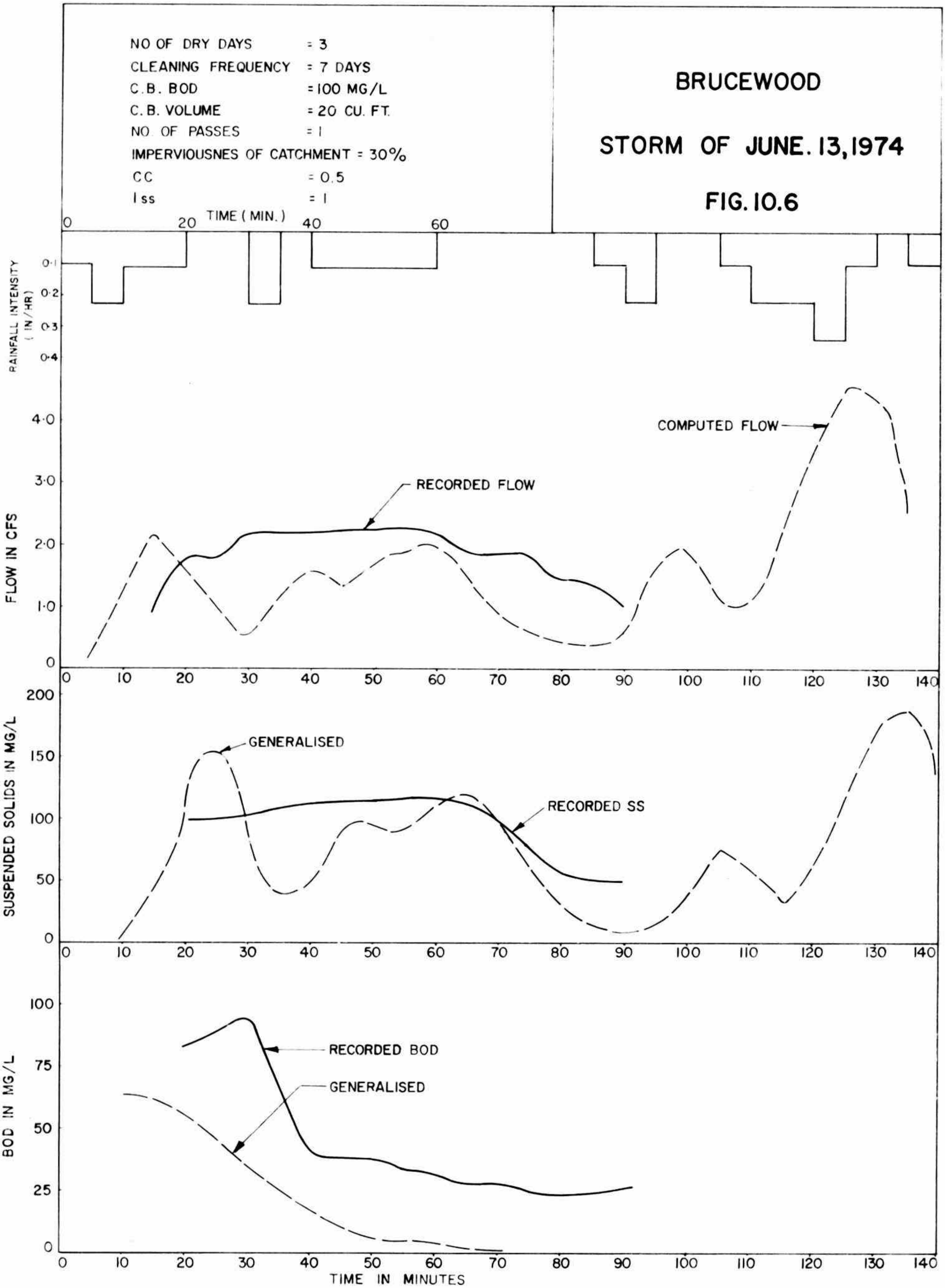


TIME (MIN.)
0
20
40
60
RAINFALL INTENSITY (IN/HR)
0·4
0·8
BRUCEWOOD
STORM OF NOV. 20,1974
FIG.10.3
12·0
9·0
6·0
3·0
0
FLOW IN CFS
COMPUTED FLOW
RECORDED FLOW
NO OF DRY DAYS = 8
CLEANING FREQUENCY = 10 DAYS
C.B. BOD = 60 MG/L
C.B. VOLUME = 20 CU. FT.
NO. OF PASSES = 1
Iss = 1
CC = 0·5
0
20
40
60
80
100
120
140
800
600
400
200
0
SUSPENDED SOLIDS IN MG/L
SWMM
RECORDED SS
GENERALISED
0
20
40
60
80
100
120
140
80
60
40
20
0
BOD IN MG/L
SWMM
RECORDED BOD
GENERALISED
0
20
40
60
80
100
120
140
TIME IN MINUTES



BRUCEWOOD
STORM OF AUG. 29, 1975
FIG. 10.4
TIME (MIN.)
RAINFALL INTENSITY (IN/HR)
0·12
0·24
0·36
FLOW (C.F.S.)
RECORDED FLOW
COMPUTED FLOW
SUSPENDED SOLIDS (MG/L)
No. OF DRY DAYS = 2
CLEANING FREQUENCY = 14 DAYS
C.B. BOD = 60 mg/l
C.B. VOLUME = 20 Cu. Ft.
No. OF PASSES = 1
Iss = 1 cc = 0.5
MEASURED DATA FOR DUST AND DIRT ACCUMULATION RATE IS USED
D.D = 0.5 lbs/day/100' of Curb
SS = 830 mg/gm D.D
SWMM
RECORDED SS
GENERALISED
BOD (MG/L)
SWMM
GENERALISED
RECORDED BOD
TIME (MIN.)


BRUCEWOOD
STORM OF AUG. 11, 1975
FIG. 10.5
RAINFALL INTENSITY (IN/HR)
0.12
0.24
0.36
FLOW IN CFS
RECORDED FLOW
COMPUTED FLOW
SUSPENDED SOLIDS IN MG/L
GENERALISED
RECORDED SS
NO OF DRY DAYS = 10
CLEANING FREQUENCY = 14 DAYS
C.B. BOD = 60 MG/L
C.B. VOLUME = 20 CU. FT.
NO. OF PASSES = 1
Iss = 1. CC = 0·5
MEASURED DATA FOR DUST AND DIRT
ACCUMULATION IS USED
D/D = 0·6 LBS/100 FT/DAY
SUSPENDED SOLIDS = 570 MG/GM OF D/D
BOD IN MG/L
GENERALISED
RECORDED BOD
TIME IN MINUTES


NO OF DRY DAYS = 3
CLEANING FREQUENCY = 7 DAYS
C.B. BOD = 100 MG/L
C.B. VOLUME = 20 CU. FT.
NO. OF PASSES = 1
IMPERVIOUSNES OF CATCHMENT = 30%
CC = 0.5
Iss = 1
BRUCEWOOD
STORM OF JUNE. 13, 1974
FIG. 10.6
TIME (MIN.)
RAINFALL INTENSITY (IN/HR)
FLOW IN CFS
COMPUTED FLOW
RECORDED FLOW
SUSPENDED SOLIDS IN MG/L
GENERALISED
RECORDED SS
BOD IN MG/L
RECORDED BOD
GENERALISED
TIME IN MINUTES

CONTINUOUS SIMULATION

CHAPTER 11

CHAPTER 11
CONTINUOUS SIMULATION

11.1 GENERAL

Single event simulation using the SWMM or other detailed models provides information on the characteristics of the stormwater runoff from a catchment for a particular rainfall event. The initial conditions must be supplied to the model. These include, the number of dry days preceding the event, the infiltration potential, the available depression storage and capacity of artificial storage facilities. Often these values have to be assumed because of insufficient data and these assumptions can often significantly influence the simulated result. The antecedent dry period controls the pollutant accumulation, (as indicated in Chapter 5), while the initial infiltration and storage conditions may alter the simulated peak flows. From the standpoint of pollution control, statistical data describing a large number of events may be more important than single low frequency design events. Therefore it is often necessary to investigate the long term runoff history and the associated pollutant accumulation and depletion. This can be accomplished using a simulation model capable of processing a long term continuous precipitation record. From the simulated series of events, it is possible to define a critical or "worst" overflow event in terms of total pollutant load, total volume, peak flow or as some combination of these.

Several continuous simulation models were reviewed for possible interface with SWMM. The STORM model was selected. This is a simple continuous simulation model having the capability of performing both quantity and quality computations and assessing the influence of different storage and treatment capacities on the number and magnitude of the overflows. This model was then tested against both the SWMM model, and against overflow records from two test watersheds. Previous studies reported in the literature did not present a comparison of the two models. An important concept

developed in this study was the use of the continuous simulation model for screening a long period of runoff events to isolate critical events for subsequent SWMM simulations. Information produced in this screening process may be used to set up initial conditions for detailed modelling. A methodology for the interfacing of continuous and single event simulation models is discussed at the end of this chapter.

11.2 REVIEW OF AVAILABLE MODELS

Continuous simulation models appear to have been first developed for rural watershed, and only recently modified for urban conditions and for quality simulation. Several available urban models were reviewed for suitability under the terms of this study. Some of these were examined in more detail and will be briefly described below. For a more detailed discussion of all long term simulation models see Brandstetter [1].

11.2.1 *Storage, Overflow and Treatment Model (STORM)*

The STORM model was developed jointly by the U.S. Corps of Engineers and Water Resources Engineers Inc. [2]. The model computes stormwater runoff from a single catchment in hourly time steps based on the record of a single raingauge. The rainfall depth in excess of the depression storage is transformed to direct runoff through the use of a specified runoff coefficient at each time step. Runoff from both pervious and impervious areas of the catchment is simulated. Snowmelt computations based upon the "degree-day" method may also be performed. The water balance between storms is determined via the recovery of depression storage based upon specified potential evapotranspiration rates.

The model performs no routing computations, and all direct runoff computed for each time step is assumed to drain from the catchment in that time step Various combinations of storage and treatment capacities may be modelled and the effect of these on stormwater overflows investigated. Quality computations may be performed in each time step based upon the pollutant

washoff from different land uses. Five common pollutants can be simulated. The quality computations are essentially the same as those performed in the SWMM. Dry weather flow is not considered. Between storms, the amounts of available surface pollutants are modified according to the number of dry days for accumulation and the number of street sweepings.

On the basis of the simulations performed in this study, the STORM model is relatively inexpensive to operate and the computer cost for processing an entire year of rainfall data is less than $10.00. The computer program is non proprietary and available from the Corps of Engineers, Hydrologic Engineering Centre.

11.2.2 *Chicago Flow Simulation Program (FSP)*

The FSP model, [3], simulates runoff from a multiple subcatchment watershed using precipitation data from one or more gauges at any time interval. Runoff from both pervious and impervious areas is simulated. Dry weather flow and snowmelt, are accounted for in computing the total outflow. The water balance between storms is also considered through the use of infiltration and evapotranspiration computations.

Routing of combined runoff through circular pipes and trapezoidal channels is performed, and reservoir storage at the outlet may be modelled. No water quality computations may be performed with the model. The computer program is available from the authors.

11.2.3 *Hydrocomp Simulation Program (HSP)*

The HSP model, Hydrocomp Inc. [4], simulates runoff from a multi-subcatchment watershed using precipitation data from one or more gauges. Short time intervals are usually employed, and runoff from both pervious and impervious areas is simulated. The runoff computations are more sophisticated than those used in most continuous simulation models. Infiltration, interception,

evapotranspiration, and depression storage are computed and the model may calibrated through adjustment of the coefficients for each of these processes.

Snow accumulation and melt are computed using the Corps of Engineers energy equations. Dry weather flow is also computed and combined with the surface runoff.

Complete routing computations are performed for sewers and channel networks using the Kinematic Wave Theory, accounting for upstream and downstream controls. Diversion and storage structures may be modelled at specific points in the network. Quality computations are performed for surface runoff, dry weather flow and constituent routing through the network. Up to 17 constituents may be modelled. The pollutant accumulation and water balance between storm events is also considered. The computer program is proprietary to Hydrocomp International Inc.

11.2.4 *Massachusetts Institute of Technology Urban Watershed Model (MIT)*

The MIT model, [5], simulates stormwater runoff for any time interval from a multi-subcatchment watershed using precipitation data from a single raingauge. Runoff from both pervious and impervious area is simulated. Evapotranspiration and infiltration are modelled during each event, but it appears that no water balance between storms is applied. Snowmelt is not modelled. Dry weather flow is computed and added to the runoff.

The Kinematic Wave Theory is used to perform routing computations in an open channel network, and information on stage and velocity is generated. No diversion control, or storage facilities are modelled. No quality computations may be performed with the model. Which is proprietary to Resource Analysis Inc., of Boston.

11.2.5 *University of Massachusetts Combined Sewer Control Simulation Model (SCM)*

The SCM program, Ray [6], simulates runoff from a multi-subcatchment watershed employing data from a single precipitation gauge. Runoff from impervious areas only is computed. Dry weather flow is computed and added to the stormwater runoff, but snowmelt is not considered. The water balance between storms is accounted for by varying the loss and depression storage functions. Hourly time steps are employed in the computations.

Flow routing is performed using the Dynamic Wave Theory which permits consideration of upstream or downstream controls, but no storage or diversion functions are modelled. No quality computations are performed in the model. The program is available from the authors.

11.3 SELECTION OF A CONTINUOUS MODEL

The selection of a continuous simulation model was based mainly on the ability of the model to perform both quantity and quality computations, and to simulate the effects of different storage and treatment capacities on stormwater overflows. Computer time and data requirements were also considered. The model selected after review of all criteria shown on the summary Table 11.1, was STORM. Although at present, this model does not account for dry weather flow, a new version is being developed which does, and which employes new runoff computations. The quality computations in STORM are essentially the same as those of SWMM. This is a desirable feature since it facilitates the use of STORM for screening a series of events for subsequent SWMM simulations. The STORM model is well documented, is freely available, and is inexpensive to run.

TABLE 11.1

LONG TERM SIMULATION MODELS

	Chicago Flow Simulation	STORM	Hydrocomp	MIT	U. of Mass.
Runoff Quality	No	Yes	Yes	No	No
Storage	Yes	Yes	Yes	Yes	No
Snowmelt	Yes	Yes	Yes	No	No
Dry Weather Flow	Yes	No	Yes	Yes	Yes
Flow Routing	Yes	No	Yes	Yes	Yes
Proprietary	No	No	Yes	Yes	No

11.4 OVERVIEW OF THE STORM MODEL

11.4.1 *Runoff Quantity*

In STORM, runoff is computed hourly based upon the average watershed runoff coefficient, the rainfall within the hour and depression storage, according to the following formulae:

$$R = C(P - f) \tag{11.1}$$

where R = urban area runoff in inches per hour

C = composite runoff coefficient dependent on urban land use

P = rainfall plus snowmelt in inches per hour over the urban area; and

f = available urban depression storage in inches per hour

The runoff generated in each hour is assumed to drain from the watershed within that hour, and may then be modified by any treatment or storage option specified.

11.4.2 *Runoff Quality*

Runoff quality is also computed in hourly time steps. The rate of removal of a pollutant from the watershed within each time step is assumed to be exponentially related to the amount remaining after the preceding step. For each of five pollutants, (Suspended Solids, Settleable Solids, BOD, Nitrogen, PO_4) the relationship is:

$$M_p = A\,P_{(t)}\,(1-e^{-E_u R_i \Delta t})/\Delta t \tag{11.2}$$

where M_p is the amount of pollutant washed off in this time step, Δt

A is the availability coefficient

$P_{(t)}$ is the amount of pollutant on the watershed at the start of this time step

R_i is the runoff rate from impervious areas

E_u is the urban washoff decay coefficient

The user may supply the various coefficients or rely on the default values in the program. Reference should be made to the User's Manual for a more detailed description of these [2]. The amount of pollutant accumulation on the watershed is governed by the number of dry days, the total length of curb and gutter, and the dust and dirt accumulation rate on the watershed. The various pollutants are expressed as fractions of the dust and dirt (as described in Chapter 5). The maximum permissible amount of pollutant is limited to that accumulated in 90 dry days. Non urban default values are discussed in Volume 2.

11.4.3 *Storage and Treatment*

The computation of the volumes of runoff stored, treated and overflowed are based upon the computed runoff and the specificed storage volume and treatment rate. No storage is used until the specified treatment rate is exceeded. If subsequently the specified storage capacity is filled, excess runoff overflows to the receiving water body. When the runoff rate is less than the treatment rate, the excess treatment capacity is used to empty storage. The pollutant load in the overflow is determined from a mass balance of the amount of a constituent in the runoff and the amounts in storage and lost to treatment. Storage and treatment are treated simply as volumetric functions.

The programme provides a description of each event and a statistical summary of all events encountered in the input record. An event is considered to begin when the treatment rate is exceeded and end when the storage unit is emptied, or when the runoff falls below the treatment rate.

11.4.4 *Other Computations and Features*

At present STORM does not simulate the dry weather flow, which for small storms may be a significant contributor to outflow quantity and quality. The "first-flush" phenomenon is primarily a result of the flushing of the accumulated solids, deposited in the sewer pipes from the DWF and from the flushing of solids and BOD in catchbasins. Consequently it may be expected that STORM will not produce a distinct "first-flush" effect.

The U.S. Army Corps of Engineers is currently preparing a new version of STORM which includes dry weather flow computations and unit hydrograph techniques [10].

The shape of the watershed is not considered by STORM, nor is the time of concentration taken into account. It is assumed that all runoff flows out of the catchment during the time step in which it is generated. For larger watersheds, with a concentration time greater than one hour, the computed hydrograph will generally occur earlier than the observed one. The reverse would be true for smaller watersheds with a concentration time less than one hour. This is not usually of great concern in many studies, however.

Snowmelt is computed using the degree-day method, according to the formula:

$$MELT = COEF \times (T - T_T) \text{ in which} \tag{11.3}$$

MELT = snowmelt in inches over the basin

COEF = degree-day coefficient, ranging from .05 to .15 inches per degree-day

T = average daily air temperature °F

T_T = temperature at which snow begins to melt

Snowmelt is computed only for those days when the average daily temperature is above the temperature at which snow begins to melt otherwise the precipitation is added to the snowpack. The computed snowmelt is distributed uniformly throughout the period, 9 a.m. to 5 p.m.

The Universal Soil Loss Equation [7,8], is used to compute soil erosion. This emperical equation was developed for agricultural land. Many empirical coefficients may require calibration. However, the method is an approximative technique which may be used to define ranges or limits of watershed sediment yield. Use of this routine is optional.

The runoff model is calibrated by varying the following runoff parameters:

(a) Runoff coefficients for pervious and impervious area
(b) Evapotranspiration rates
(c) Depression storage
(d) Rainfall reduction factor used to relate point precipitation to basin average rainfall

Because the model considers the entire watershed as a single computational catchment, it is quite sensitive to each of these parameters and rapid calibration for period totals is generally possible.

11.5 APPLICATION OF STORM TO TEST WATERSHEDS

STORM was applied to two real test areas on which the SWMM had been previously applied for other parts of this study. The objective was to verify the STORM results over the longer period against measured overflows and to make a comparison between the detailed SWMM simulation and the STORM results for selected storms.

11.5.1 *Application of Storm to Test Watersheds*

The West Toronto Urban Watershed was selected for STORM simulation because overflow measurements were available for a relatively long period and because detailed SWMM simulations had been conducted. The period of April 1, 1973 to October 31, 1973 was simulated by processing the hourly rainfall record from the Old Weston Road gauge, situated roughly in the middle of the watershed. Quality simulation was not carried out since no measurements are available.

The program was calibrated by making slight adjustments to the depression storage, the runoff coefficient, and the rainfall reduction factor relating basin average rainfall to gauge rainfall. The program was then run for the entire 7-month period. The results are summarized in Table 11.2. The measurements have been corrected to account for intercepted and diverted flows. No storage or treatment was considered in the STORM analysis.

This table indicates a reasonably good agreement between simulations and measurements. The apparent extra duration of computed overflows could be attributed to the fact that STORM totals are computed in hourly increments whereas measured durations can be accumulated in partial hours or minutes. Thus a 1/2 hour rainfall may produce a measured 1/2 hour overflow, but STORM would assign this a 1 hour duration.

TABLE 11.2

COMPARISON OF MEASURED AND COMPUTED OVERFLOWS FOR THE WEST TORONTO AREA

	Measured			Computed with STORM		
1973	Volume Inches	No. of Events	Duration Hours	Volume Inches	No. of Events	Duration Hours
April	.55	7	17	.69	7	46
May	1.17	11	28	1.52	10	48
June	.77	9	17	.82	10	19
July	.74	6	20	.69	7	16
August	1.34	5	15	.99	4	11
September	.76	7	19	.69	6	20
October	1.65	8	65	1.95	10	67
Total	6.98	53	181	7.36	54	227

A comparison between the measured hydrograph, the SWMM computed hydrograph (produced under other sections of this study) and the STORM computed runoff was made for five selected events, covering a range of rainfall intensities and storm durations. The comparisons were based on the total runoff hydrograph, that is the measured overflow hydrograph adjusted for intercepted flow at the outlet of the West Toronto area, (see Table 11.3).

TABLE 11.3

COMPARISON OF MEASURED AND COMPUTED RUNOFF FOR THE WEST TORONTO AREA

1973	Measured Q_p CFS	Measured Vol. Inches	SWMM Q_p CFS	SWMM Vol. Inches	STORM Q_p CFS	STORM Vol. Inches
May 10	893	.26	740	.27	707	.46
June 22	703	.13	673	.15	374	.15
August 1	1064	.69	816	.81	928	.89
Sept. 23	383	.19	367	.18	202	.06
Oct. 2	509	.12	457	.11	368	.18
Total		1.39		1.52		1.74

The SWMM results generally compare better to the measurements than do those of STORM. On September 23, October 2, June 22, and May 10, the peak flows computed by SWMM are closer to the measured peaks than those of STORM. On August 1, the heavy surcharging computed by SWMM resulted in a significant reduction in peak flow, since the SWMM simply stores flow in excess of pipe capacity. The total volumes computed using SWMM for the five events compares reasonably with the measured volumes, but those produced by STORM are higher. This comparison suggests that STORM compares fairly well with SWMM in terms of total volume. Figure 11-1 shows a comparison between the measured, SWMM and STORM hydrographs for these events.

11.5.2 *Bannatyne Test Area (Winnipeg)*

STORM was applied to this test area for the four month period, June to September 1971. Measurements of both quantity and quality of overflow are available. However, it has been noted in Chapter 5 that DWF deposits control the pollutant loads and so the use of STORM for quality simulation is likely to be somewhat inaccurate. The area has also been modelled using the SWMM, for selected storms during 1971. Table 11.4 compares the total observed and computed runoff for each of the four months. In

this case, a good agreement between the total number and volume of overflows was obtained. The computed durations were less than those observed.

TABLE 11.4

COMPARISON OF MEASURED AND COMPUTED OVERFLOWS FOR THE BANNATYNE AREA

Month (1971)	Measured			Computed by STORM		
	Volume Inches	Number	Duration Hours	Volume Inches	Number	Duration Hours
June	.47	9	26	.52	9	20
July	1.17	12	51	.93	12	34
Aug.	.27	2	11	.22	2	9
Sept.	.26	1	4	.24	1	3
Total	2.17		92	1.92		66

The results of the detailed simulations using SWMM are shown in Table 11.5 along with the corresponding STORM results. Both models produced a reasonably good simulation of the runoff volume for six events. The total runoff volume computed by STORM agrees with the measured total somewhat better than that computed using SWMM. Figure 11-2 compares the hydrographs computed by STORM to the recorded flows for the three major events in the study period.

TABLE 11.5

COMPARISON OF MEASURED AND COMPUTED HYDROGRAPHS FOR THE BANNATYNE AREA

Month (1971)	Measured Qp CFS	Measured Volume Inches	SWMM Qp CFS	SWMM Volume Inches	STORM Qp CFS	STORM Volume Inches
June 19	47	.15	31	.15	37	.18
July 3	60	.12	48	.07	27	.05
July 15	15	.02	8	.04	17	.05
July 17	32	.10	22	.10	27	.11
July 28	35	.10	30	.08	27	.09
Sept. 5	100	.26	95	.19		.24
Total		.80		.63		.72

Direct comparisons of total pollutant loads were not possible because STORM does not consider dry weather flow. However, a comparison of the total pollutant emissions computed by SWMM and STORM for surface runoff only, initially showed large discrepancies. This was due to the fact that street sweeping is only applied in STORM when the number of dry days exceeds the cleaning frequency. Thus, when using the reported frequency of 42 days, no cleaning occurs unless there is a dry period in excess of 42 days, and consequently pollutants build up to maximum values on the watershed in a short time. The simulation was repeated with a short cleaning interval, which permitted the accumulated surface load to be cleaned more realistically. A better agreement between SWMM and STORM surface runoff pollutant computations is evident in the simulation with the shorter cleaning interval. The amount of pollutants allowed to accumulate was greatly reduced by sweeping, resulting in smaller total discharge.

Table 11.6 shows the comparison between total pollutant loads in the runoff using both the SWMM and STORM. The SWMM simulations were based on the actual number of dry days.

TABLE 11.6

COMPARISON OF S.S. DISCHARGES COMPUTED BY STORM AND SWMM FOR THE BANNATYNE AREA

Month (1971)	Suspended Solids in Pounds — STORM: 42 Day Sweeping Interval	STORM: 13 Day Sweeping Interval	SWMM: Runoff Only
July 19	38164	2253	2710
July 15	6754	850	658
July 17	15043	2738	2112
July 28	14600	3336	2931
Sept. 5	93824	4234	15195
Total	168385	13411	23606

11.6 CONTINUOUS SIMULATION USING "LUMPED" SWMM

The encouraging results obtained using the simplified "lumped" SWMM are discussed in Chapter 9. It has been demonstrated that the SWMM may be operated in a simplified manner with respect to both area and time (larger time steps). The conclusion of these results was that the application of the SWMM could be extended from the simulation of the single-event to multi-event simulation. Because of reduced data preparation and computer storage and computer time required, many simulations of pre-selected events can be accomplished at a cost comparable with that of detailed single-event simulation. Initial watershed conditions are still required for each simulation.

A further extension of this aspect of the SWMM application will be possible when the model is modified to account for the water balance between storms. This may be done by re-establishing the soil infiltration capacity between storms and also by accounting for the loss of depression storage. This will permit the correct antecedent moisture conditions to be modelled continuously and will result in appropriate initial conditions for each storm

in the continuous record. In order to model quality continuously, it is necessary to allow the pollutants to accumulate during dry periods. These modifications would permit the SWMM to be operated in a continuous fashion with a reasonable degree of accuracy.

The advantages of using the SWMM for continuous simulation depend upon the level of accuracy required in the results. The SWMM provides more detailed storage/treatment options and information than does STORM. The SWMM also allows for cost estimation which is an important feature in evaluating alternatives. However, STORM provides the basic data which is necessary in preliminary studies of un-gauged urban systems. Normally the primary concern of municipalities when evaluating their drainage systems is the overflow magnitude and frequency.

Current research at the University of Florida [9], is directed toward making these and other modifications to enable SWMM to be extended for continuous simulation. Initial results are encouraging, and further modification and testing of the continuous simulation SWMM should be continued.

11.7 INTERFACING CONTINUOUS AND SINGLE-EVENT SIMULATION MODELS

The proposed methodology consists of three stages:- data preparation, planning stage and design/analysis stage. The data preparation stage is based on the Data Analysis Program discussed in Chapter 12. The planning stage is based on the STORM model and partly on a lumped and modified version of the SWMM (or the Generalized Quality Model described in Chapter 10, for surface runoff applications). For the design/analysis stage, the SWMM or the WRE Model is used. The methodology is schematically outlined in Figure 11.3. The description of individual components and of their application is presented in the following sections.

11.7.1 *Preparation Stage*

The application of most urban runoff models usually requires a large volume of data and the cost of preparation of these data can amount to a significant portion of the total study costs. Therefore, it is desirable to simplify and computerize this part of a study to the maximum possible extent. Climatological data are typically most voluminous data involved in urban runoff studies. Their processing has been computerized by means of the Data Analysis Program (DAP), described in Chapter 12. The DAP serves as an interface between the existing data banks and both the planning and design stage models. For the planning stage, hourly precipitation and temperature data are required. These are available from the Atmospheric Environment Service (AES).

The output from the DAP Model consists of the punched cards required for STORM and SWMM models and of event summaries. These summaries list the clock time of the onset and end of each storm, its duration, the total depth of rainfall, the peak intensity and the antecedent dry period. The event summaries are useful for a rapid review of the precipitation data, and eventually, for the identification of critical rainfall/runoff events, where this task is not done using the STORM model.

The remaining physical and operational input data describing the runoff model parameters are processed manually. In order to assess the importance of individual parameters in the SWMM model, sensitivity analyses and level of catchment discretization have been investigated (Chapter 4 and 9).

11.7.2 *Planning Stage*

In the planning stage, various land use alternatives, drainage systems and the resultant pollutional impacts on the receiving waters are evaluated. Typically, at this stage only limited information describing the watershed is available, and consequently, a detailed runoff simulation would not be feasible or appropriate. At the same time, it is important to establish the probability of occurrence of runoff events of specific magnitudes.

This can be achieved by continuous simulation of the urban runoff over a long period using STORM.

The SWMM model would next be applied in a simplified (lumped) manner for a limited number of critical precipitation/runoff events, the frequencies and antecedent conditions of which would have been determined previously by STORM. If the planning only involves surface runoff simulation the Generalized Quality Model may be used in lieu of the lumped SWMM for quality simulation.

At the planning stage, the SWMM can be applied in various degree of detail depending on the purpose of the study. Work described in Chapter 9 indicates that the SWMM can be applied as a spatially lumped model without a significant sacrifice in the accuracy of simulation. Reduction of the number of elements in the RUNOFF and TRANSPORT blocks to a minimum, results in a considerable saving in data reduction time and in computer time requirements. Application of the lumped SWMM model for the Bannatyne and Toronto West areas, showed that it was possible to represent both areas as single aggregated catchments, in spite of their relatively large areas.

The long term effectiveness of various alternative storage volumes and treatment capacities is broadly assessed using STORM. A more detailed analysis is possible using the STORAGE block of the SWMM, which can simulate both in-line and off-line storage. The SWMM also allows the simulation of various treatment processes.

Consequently, in a comprehensive planning study the role of STORM should usually be limited to that of screening for identification of critical events, which are subsequently simulated using SWMM.

Once the hydrographs and pollutographs have been simulated by the SWMM, different runoff (or overflow) control alternatives may be studied. The SWMM can consider seven levels of treatment and estimates, very approximately, the costs of implementing these control alternatives. In some instances, a preliminary analysis of the impact of critical discharges to the receiving waters would also be carried out in the planning phase.

At the completion of the planning stage, the user has a good indication of the nature of the runoff or overflow problem in the study area and also has a feeling for the effectiveness of various runoff/overflow control measures for a number of critical rainfall/runoff events. The information obtained during this phase of a study is at a planning level, for which the relative effects and magnitudes are more important than the absolute values required for subsequent design purposes.

11.7.3 *Design Analysis Stage*

In this stage, the design of a drainage system and pollution control facilities is carried out, as well as a detailed study of probable impacts in the receiving waters. Consequently, it is desirable to produce accurate hydrographs and pollutographs for selected events using a calibrated, detailed simulation model. At this level, SWMM would be used at a high level of discretization.

Sewer surcharging would not have been considered at the planning stage, as sewers are considered as an open channel network at that stage. Critical events for pollution abatement are not usually the low frequency storms used in design. However, surcharging, becomes very important when analyzing methods to reduce flooding in an existing sewer system of insufficient capacity, or when evaluating the response of a drainage system to a storm of a frequency lower than the design frequency. Under such circumstances, the SWMM is not applicable and other more sophisticated models are required. For simulation of backwater effects which might occur in the sewer system, it becomes necessary to use a model with dynamic wave routing, such as the WRE Model or Dorsch HVM.

Once calibrated hydrographs and pollutographs have been obtained from the detailed model, the storage and treatment design can be finalized, following an evaluation of the environmental impact of the effluent or overflow on receiving waters.

11.7.4 *Case Study*

The hypothetical case study used data from the Bannatyne combined sewer district for which overflow records for four months were available. An abbreviated output from the STORM model is shown in Table 11.7. A visual inspection of the input indicated two storms of particular interest - Storm No. 11 (July 7, 1971), from the quantity point of view, and Storm No. 24 (Sept. 5, 1971) from the quality point of view. The latter storm not only produced a significant runoff, but also had the longest antecedent dry period, 16 days. Such a dry spell would allow a considerable accumulation of pollutants prior to the storm. Storm No. 24 was modelled using the calibrated SWMM. The results of this simulation are shown in Figure 11-4. In the same Figure, the reduction in the SS-pollutograph resulting from the control of runoff employing storage, swirl concentration, and microstraining (treatment level 4 in the SWMM) are also shown with the corresponding costs. Finally, it was assumed that the catchment discharges into a lake and a receiving water body simulation was conducted. Suspended Solids were simulated at a point near the drainage outlet and the results are shown in Figure 11-5. For an antecedent dry period of 16 days with no effluent treatment, the S.S. concentrations were as high as 610 mg/l. This value was reduced to 460 mg/l for an 8 day dry period and to 60 mg/l for only one dry day. For comparison, the level for treatment (microstrainer) reduces the maximum suspended solids concentration in the first case to 210 mg/l. This raises some interesting questions regarding other possible pollution controls - e.g., interception of the dry weather flow (say 3 DWF) with and without the treatment of the overflow, or flushing of sewers with diversion of the flushings to the waste treatment plant. Such control alternatives could also be studied at the design analysis stage.

11.8 SUMMARY AND CONCLUSIONS

(a) A review of available continuous simulation models has been carried out, and the STORM model selected as being the most suitable model for application under the terms of this study.

(b) STORM has been tested against measured data from two watersheds, and against the results of detailed simulations by SWMM. The model was found to give reasonably good estimates of the overall magnitudes and frequencies of overflows. In some instances, close simulation of measured hydrographs is also achieved. However, this is not a principal objective of the application of STORM.

(c) For surface runoff, STORM may be calibrated to produce similar total pollutant loads to those computed by the SWMM over a given series of events. The state of the art of quality modelling is such that emphasis should be placed on the overall pollutant discharge rather than on instantaneous concentration values. In this respect STORM may be used to provide a useful summary of runoff quality over a long period.

(d) The present lack of a dry weather flow routine in STORM precludes any accurate characterization of combined sewer pollutant discharges. In this case a lumped version of the SWMM may be used as an intermediate planning model following the identification of critical events by STORM. It is noted that a version of STORM incorporating dry weather flow will be available in 1976.

(e) A methodology for interfacing STORM and SWMM has been described and been shown to be applicable to a simple case study.

TABLE 11.7

PAGE 1

THE BANNATYNE DISTRICT
QUANTITY ANALYSIS

TREATMENT RATE 0.0000 IN/HR, 0.0 CFS, 0.0 MGD
STORAGE CAPACITY 0.0000 INCHES, 0.0 AC-FT, 0.000 MG

WEIGHTED AVERAGE VALUES
BANNATYNE

EVENT	---D A T E--- YEAR	MO	DY	HR	HRS NO STOPAG	-R A I N F A L L- DURTN	HRS	QUANTY	RUNOFF INCHES	HRS TO EMPTY	--STORAGE-- DURTN	MAX	----O V E R F L O W---- NO	ST	DUR	WASTE	INITL	---TREATMENT---- HRS	QANTY	--AGE OF STORAGE--- AGE1	AGE2	AGE3	AGE4	AGE5
****1	********	**2	*3		*****4	****5	***6	*****7	****7A	*****8	****9	***10	*11	*12	*13	***14	***15	***16	***17	**18	**19	**20	**21	**22
1	71	6	4	13	155	1	1	0.09	0.02	1	2	0.0	1	1	1	0.01	0.01	2	0.0	0.5	0.5	0.5	0.5	0.5
2	71	6	5	3	12	2	2	0.23	0.09	1	3	0.0	2	1	2	0.09	0.09	3	0.0	0.5	1.5	1.5	1.0	1.0
3	71	6	5	17	11	2	2	0.15	0.05	1	3	0.0	3	1	2	0.04	0.04	3	0.0	0.5	1.5	1.5	1.0	1.0
4	71	6	10	5	105	3	3	0.15	0.06	1	4	0.0	4	1	3	0.05	0.05	4	0.0	0.5	2.5	2.5	1.5	1.5
5	71	6	11	24	39	2	2	0.07	0.02	1	3	0.0	5	1	2	0.01	0.01	3	0.0	0.5	1.5	1.5	1.0	1.0
6	71	6	19	11	176	4	4	0.45	0.18	1	5	0.0	6	1	4	0.18	0.17	5	0.0	0.5	3.5	3.5	2.0	2.0
7	71	6	26	4	156	2	2	0.21	0.07	1	3	0.0	7	1	2	0.07	0.07	3	0.0	0.5	1.5	1.5	1.0	1.0
8	71	6	27	19	36	2	2	0.07	0.01	1	3	0.0	8	1	2	0.01	0.01	3	0.0	0.5	1.5	1.5	1.0	1.0
9	71	6	30	14	64	2	2	0.10	0.02	1	3	0.0	9	1	2	0.02	0.02	3	0.0	0.5	1.5	1.5	1.0	1.0
10	71	7	3	19	74	1	1	0.16	0.05	1	2	0.0	10	1	1	0.04	0.04	2	0.0	0.5	0.5	0.5	0.5	0.5
11	71	7	7	1	76	4	4	0.61	0.25	1	5	0.0	11	1	4	0.25	0.23	5	0.0	0.5	3.5	3.5	2.0	2.0
12	71	7	10	3	69	4	4	0.30	0.12	1	5	0.0	12	1	4	0.11	0.10	5	0.0	0.5	3.5	3.5	2.0	2.0
13	71	7	15	20	132	3	2	0.13	0.05	1	4	0.0	13	1	2	0.05	0.05	4	0.0	0.6	0.5	0.5	0.5	0.5
14	71	7	17	21	45	6	5	0.31	0.11	1	7	0.0	14	1	5	0.11	0.08	7	0.0	0.7	2.5	2.5	1.3	1.3
15	71	7	19	7	27	2	2	0.11	0.03	1	3	0.0	15	1	2	0.02	0.02	3	0.0	0.5	1.5	1.5	1.0	1.0
16	71	7	22	13	75	2	2	0.14	0.04	1	3	0.0	16	1	2	0.04	0.04	3	0.0	0.5	1.5	1.5	1.0	1.0
17	71	7	24	3	35	2	2	0.09	0.02	1	3	0.0	17	1	2	0.01	0.01	3	0.0	0.5	1.5	1.5	1.0	1.0
18	71	7	25	6	24	2	2	0.23	0.09	1	3	0.0	18	1	2	0.09	0.09	3	0.0	0.5	1.5	1.5	1.0	1.0
19	71	7	27	17	56	2	2	0.09	0.02	1	3	0.0	19	1	2	0.02	0.02	3	0.0	0.5	1.5	1.5	1.0	1.0
20	71	7	28	20	72	4	3	0.25	0.09	1	5	0.0	20	1	3	0.09	0.09	5	0.0	0.7	1.5	1.5	0.8	0.8
21	71	7	31	5	7	2	2	0.16	0.06	1	3	0.0	21	1	2	0.06	0.06	3	0.0	0.5	1.5	1.5	1.0	1.0
22	71	8	16	21	394	4	4	0.34	0.13	1	5	0.0	22	1	4	0.13	0.11	5	0.0	1.0	2.5	2.5	1.0	1.1
23	71	8	19	8	54	5	5	0.21	0.09	1	6	0.0	23	1	5	0.09	0.06	6	0.0	0.5	4.5	4.5	2.5	2.5
24	71	9	5	5	399	3	3	0.57	0.24	1	4	0.0	24	1	3	0.23	0.23	4	0.0	0.5	2.5	2.5	1.5	1.5
AVE OF 24 EVENTS					0.0**	2.8	2.6	0.22	0.08	1.0	3.8	0.00		0.0*				3.8	0.00	1.1	1.9	1.9	1.2	1.2
AVE OF 24 OVRFLW EVENTS						2.8	2.6	0.22	0.08	1.0	3.8	0.0 *		1.0	2.6	0.08	0.08	3.8	0.00	0.5	1.9	1.9	1.2	1.2

* NON-OVERFLOW EVENTS ONLY.
EXCLUDING* DRY PERIODS

AVERAGE ANNUAL STATISTICS FOR 1 YEARS OF RECORD FOR THE PERIOD BEGINNING 710604 AND ENDING 710905

NUMBER OF EVENTS 24.0

NUMBER OF OVERFLOWS 24.0

INCHES

TOTAL PRECIPITATION ON WATERSHED 5.47

TOTAL RUNOFF FROM WATERSHED 1.92 FRACTION OF RAINFALL 0.35

OVERFLOW TO RECEIVING WATER 1.92 FRACTION OF RAINFALL 0.35, OF RUNOFF 1.00

TABLE 11.7 (cont'd.)

1PAGE 1

THE BANNATYNE DISTRICT
QUALITY ANALYSIS

TREATMENT RATE 0.0000 IN/HR, 0.0 CFS, 0.0 MGD
STORAGE CAPACITY 0.0000 INCHES, 0.0 AC-FT, 0.000 MG

WEIGHTED AVERAGE VALUES
BANNATYNE

EVENT	EVENT BEGAN YR	MO	DY	HR	RAIN FALL INCHS	STORM RUNOFF INCHS QANTY	STORM RUNOFF TOTAL POUNDS SUSP	SETL	BOD	N	PO4	STORAGE OVERFLOW SEQ NO	INCHS QANTY	TOTAL POUNDS SUSP	SETL	BOD	N	PO4	FIRST 3 HOURS OVERFLOW INCH Q	TOTAL POUNDS SUSP	SETL	BOD	N	PO4
****1	*******			2 *3	****4	****5	*****6	****7	****8	****9	**10	*11	***12	****13	***14	***15	***16	**17	**18	***19	**20	**21	**22	**23
1	71	6	4	13	0.09	0.02	418	2	74	23	2	1	0.01	418	2	74	23	2	0.01	418	2	74	23	2
2	71	6	5	3	0.23	0.09	3278	14	439	170	17	2	0.09	3278	14	439	170	17	0.09	3278	14	439	170	17
3	71	6	5	17	0.15	0.05	1227	6	158	63	6	3	0.04	1226	6	158	63	6	0.04	1226	6	158	63	6
4	71	6	10	5	0.15	0.06	2578	13	346	134	14	4	0.05	2578	13	346	134	14	0.05	2578	13	346	134	14
5	71	6	11	24	0.07	0.02	661	4	93	35	4	5	0.01	660	4	93	35	4	0.01	660	4	93	35	4
6	71	6	19	11	0.45	0.18	19115	92	2231	975	99	6	0.18	19114	92	2231	975	99	0.17	18718	89	2185	955	97
7	71	6	26	4	0.21	0.07	8478	44	989	432	44	7	0.07	8478	44	989	432	44	0.07	8478	44	989	432	44
8	71	6	27	17	0.07	0.01	806	7	104	42	4	8	0.01	806	7	104	42	4	0.01	806	7	104	42	4
9	71	6	30	14	0.10	0.02	1789	12	229	92	9	9	0.02	1788	12	229	92	9	0.02	1788	12	229	92	9
10	71	7	3	19	0.16	0.05	7234	37	842	370	37	10	0.04	7253	37	842	370	37	0.04	7253	37	842	370	37
11	71	7	7	1	0.61	0.25	47785	312	5062	2408	242	11	0.25	47784	312	5062	2408	242	0.23	46245	299	4900	2331	234
12	71	7	10	3	0.30	0.12	11687	89	1253	590	59	12	0.11	11686	89	1253	590	59	0.10	10905	81	1169	550	55
13	71	7	15	20	0.13	0.05	5239	40	613	267	27	13	0.05	5239	40	613	267	27	0.05	5239	40	613	267	27
14	71	7	17	21	0.31	0.11	11886	92	1320	603	61	14	0.11	11885	92	1320	603	61	0.08	9388	69	1045	476	49
15	71	7	19	7	0.11	0.03	2032	20	228	103	10	15	0.02	2031	20	228	103	10	0.02	2031	20	228	103	10
16	71	7	22	13	0.14	0.04	4605	35	520	234	24	16	0.04	4605	35	520	234	24	0.04	4605	35	520	234	24
17	71	7	24	3	0.09	0.02	1358	14	164	70	7	17	0.01	1358	14	164	70	7	0.01	1358	14	164	70	7
18	71	7	25	6	0.23	0.09	12277	88	1334	621	62	18	0.09	12277	88	1334	621	62	0.09	12277	88	1334	621	62
19	71	7	27	17	0.09	0.02	1975	19	227	101	10	19	0.02	1974	19	227	101	10	0.02	1974	19	227	101	10
20	71	7	28	20	0.25	0.09	12432	92	1336	628	63	20	0.09	12431	92	1336	628	63	0.09	12431	92	1336	628	63
21	71	7	31	6	0.16	0.06	8296	65	896	419	42	21	0.06	8295	65	896	419	42	0.06	8295	65	896	419	42
22	71	8	16	21	0.34	0.13	29268	201	3367	1491	151	22	0.13	29267	201	3367	1491	151	0.11	27744	186	3169	1413	143
23	71	8	19	8	0.21	0.09	11894	105	1339	604	61	23	0.09	11893	105	1339	604	61	0.06	9473	60	1070	481	49
24	71	9	5	5	0.57	0.24	88547	716	9455	4468	449	24	0.23	88545	716	9454	4468	449	0.23	88545	716	9454	4468	449
AVE OF	24 EVNTS				0.22	0.08	12267	88	1359	623	63													
AVE OF	24 OVRFLS				0.22	0.08	12287	88	1359	623	63		0.08	12286	88	1359	623	63	0.08	11905	85	1317	603	61

AVERAGE ANNUAL STATISTICS FOR 1 YEARS OF RECORD FOR THE PERIOD BEGINNING 710604 AND ENDING 710905

	SUSP	SETL	BOD	N	PO4
TOTAL POUNDS WASHOFF FROM WATERSHED	294883	2118	32616	14943	1506
TOTAL POUNDS OVERFLOW TO RECEIVING WATER	294867	2119	32616	14943	1504
CONCENTRATION OF POLLUTANTS IN OVERFLOW TO RECEIVING WATER MG/L	1252.06	9.00	138.49	63.45	6.39
FRACTION OF TOTAL WASHOFF OVERFLOWING TO RECEIVING WATER	1.00	1.00	1.00	1.00	1.00
FRACTION OF TOTAL WASHOFF INITIALLY OVERFLOWING TO RECEIVING WATER	0.97	0.96	0.97	0.97	0.97

REFERENCES - CHAPTER 11

1. Brandstetter, A., "Assessment of Mathematical Models for Storm and Combined Sewer Management", Preliminary Report, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, Ohio, August 1975.

2. U.S. Corps of Engineers. Urban Runoff: Storage, Treatment and Overlfow Model "STORM". U.S. Army, Davis, California Hydrologic Engineering Center Computer Program 723-S8-L2520, May 1974.

3. Lanyon, R.F., and J.P. Jackson. A Streamflow Model for Metropolitan Planning and Design. American Society of Civil Engineers, Urban Water Resources Research Program, Technical Memorandum No. 20, January 1974.

4. Hydrocomp International, Inc. Hydrocomp Simulation Programming-Operations Manual. Palo Alto, California, February 1972.

5. Harley, B.M., F.E, Perkins, and P.S. Eagleson. A Modular Distribution of Catchment Dynamics. Massachusetts Institute of Technology, Cambridge, Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Report No. 133, December 1970.

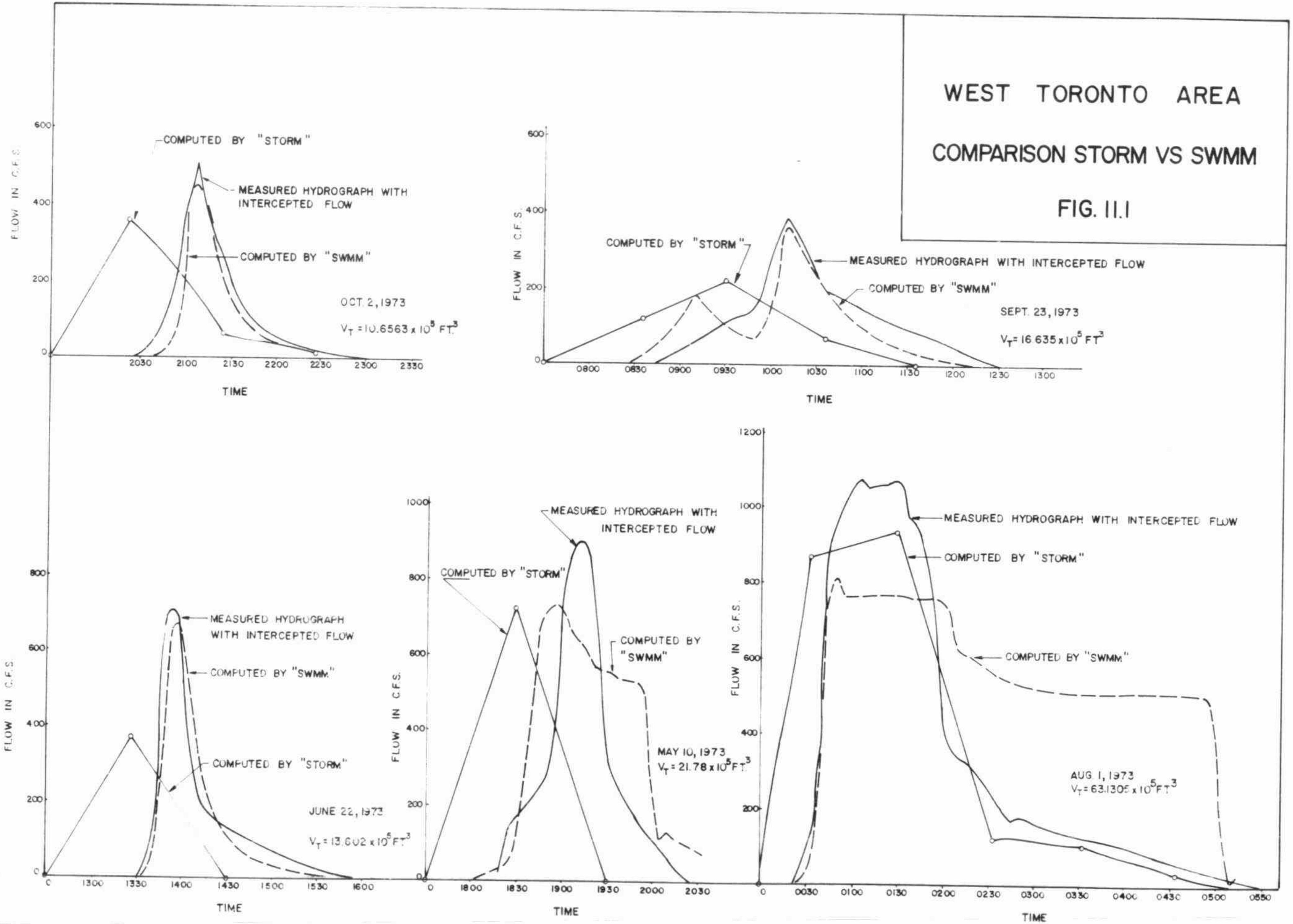
6. Ray, D.L. Simulation of Control Alternatives for Combined Sewer Overflows. University of Massachusetts, Amherst, Department of Civil Engineering, Report No. EVE 33-73-4, April 1973.

7. "Rainfall - Erosion Losses from Cropland East of the Rocky Mountains", Agriculture Handbook No. 282, Government Printing Office, Washington, DC, 1965.

8. Wischmeier, Walter H. and Dwight D. Smith, "Rainfall Energy and Its Relationship to Soil Loss", Transactions, American Geophysical Union, Vol. 39, No. 2, April 1958.

9. Smith, G.F., "Adaptation of the EPA Storm Water Management Model for use in Preliminary Planning for Control of Urban Storm Runoff", M. Eng. Thesis, University of Florida, May 1975.

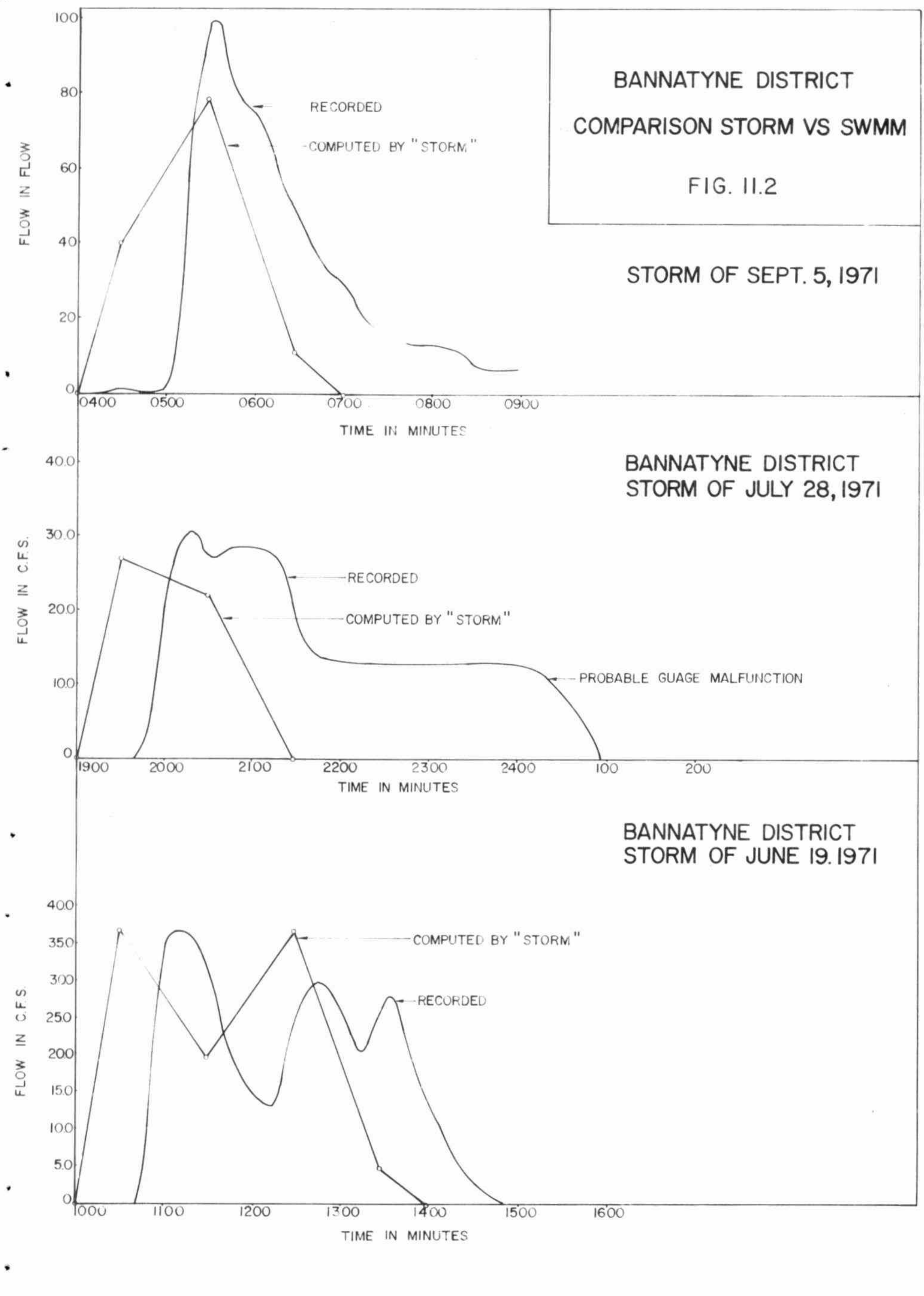
10. Abbott, J.E., Hydrologic Engineering Centre, Davis, California, Personal Communication.



WEST TORONTO AREA

COMPARISON STORM VS SWMM

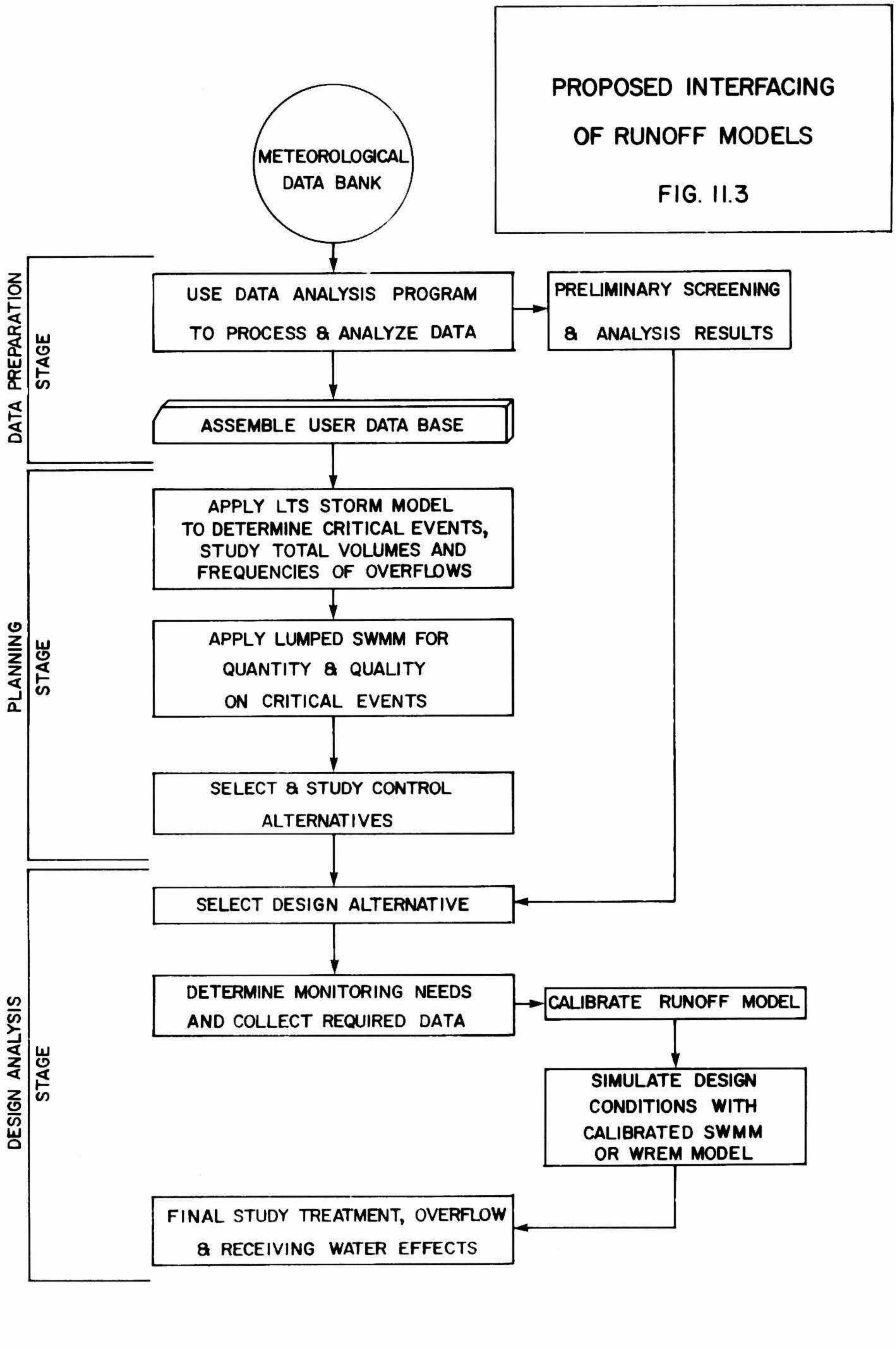
FIG. 11.1

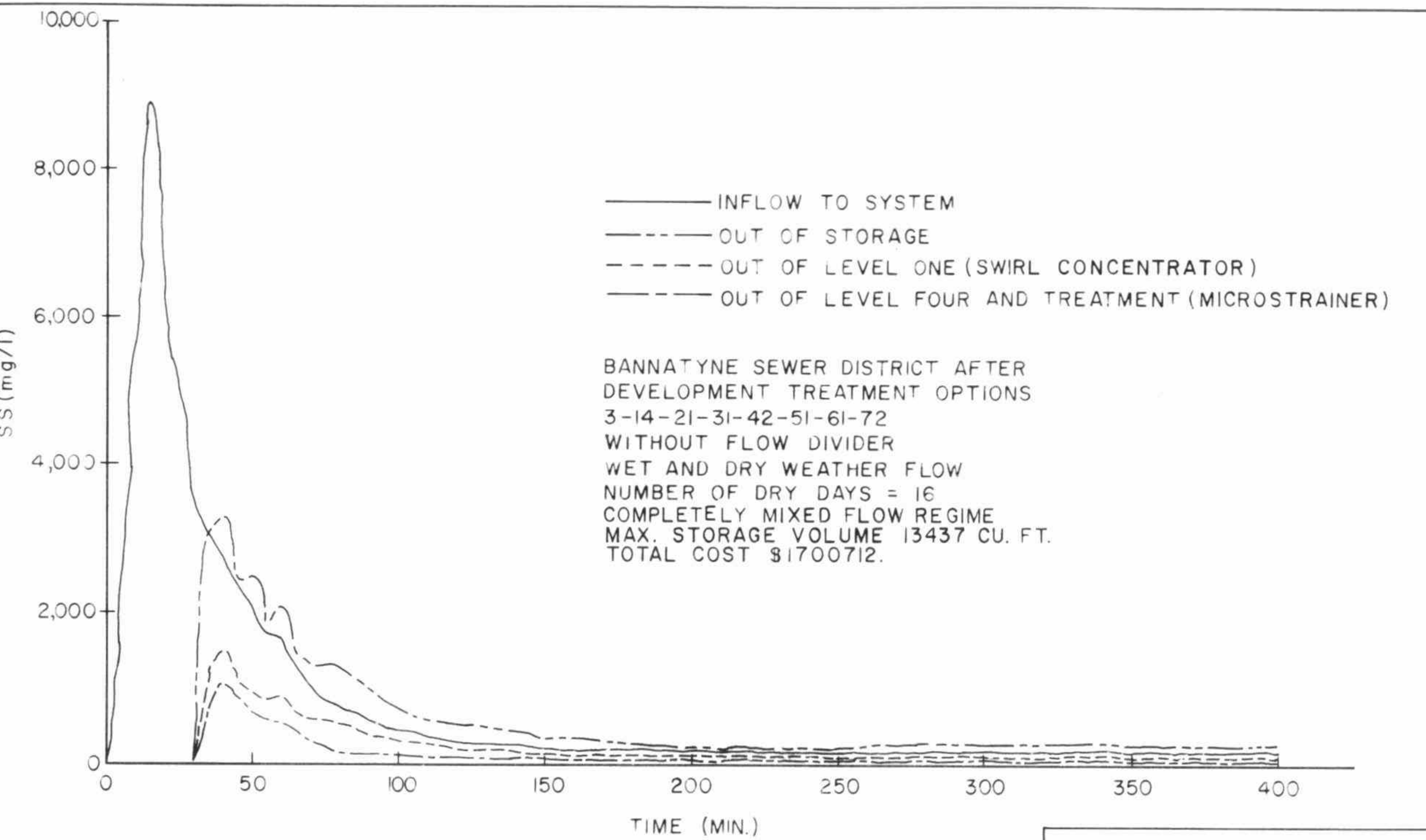


BANNATYNE DISTRICT

COMPARISON STORM VS SWMM

FIG. 11.2


PROPOSED INTERFACING
OF RUNOFF MODELS
FIG. 11.3
METEOROLOGICAL DATA BANK
DATA PREPARATION STAGE
USE DATA ANALYSIS PROGRAM TO PROCESS & ANALYZE DATA
PRELIMINARY SCREENING & ANALYSIS RESULTS
ASSEMBLE USER DATA BASE
PLANNING STAGE
APPLY LTS STORM MODEL TO DETERMINE CRITICAL EVENTS, STUDY TOTAL VOLUMES AND FREQUENCIES OF OVERFLOWS
APPLY LUMPED SWMM FOR QUANTITY & QUALITY ON CRITICAL EVENTS
SELECT & STUDY CONTROL ALTERNATIVES
DESIGN ANALYSIS STAGE
SELECT DESIGN ALTERNATIVE
DETERMINE MONITORING NEEDS AND COLLECT REQUIRED DATA
CALIBRATE RUNOFF MODEL
SIMULATE DESIGN CONDITIONS WITH CALIBRATED SWMM OR WREM MODEL
FINAL STUDY TREATMENT, OVERFLOW & RECEIVING WATER EFFECTS



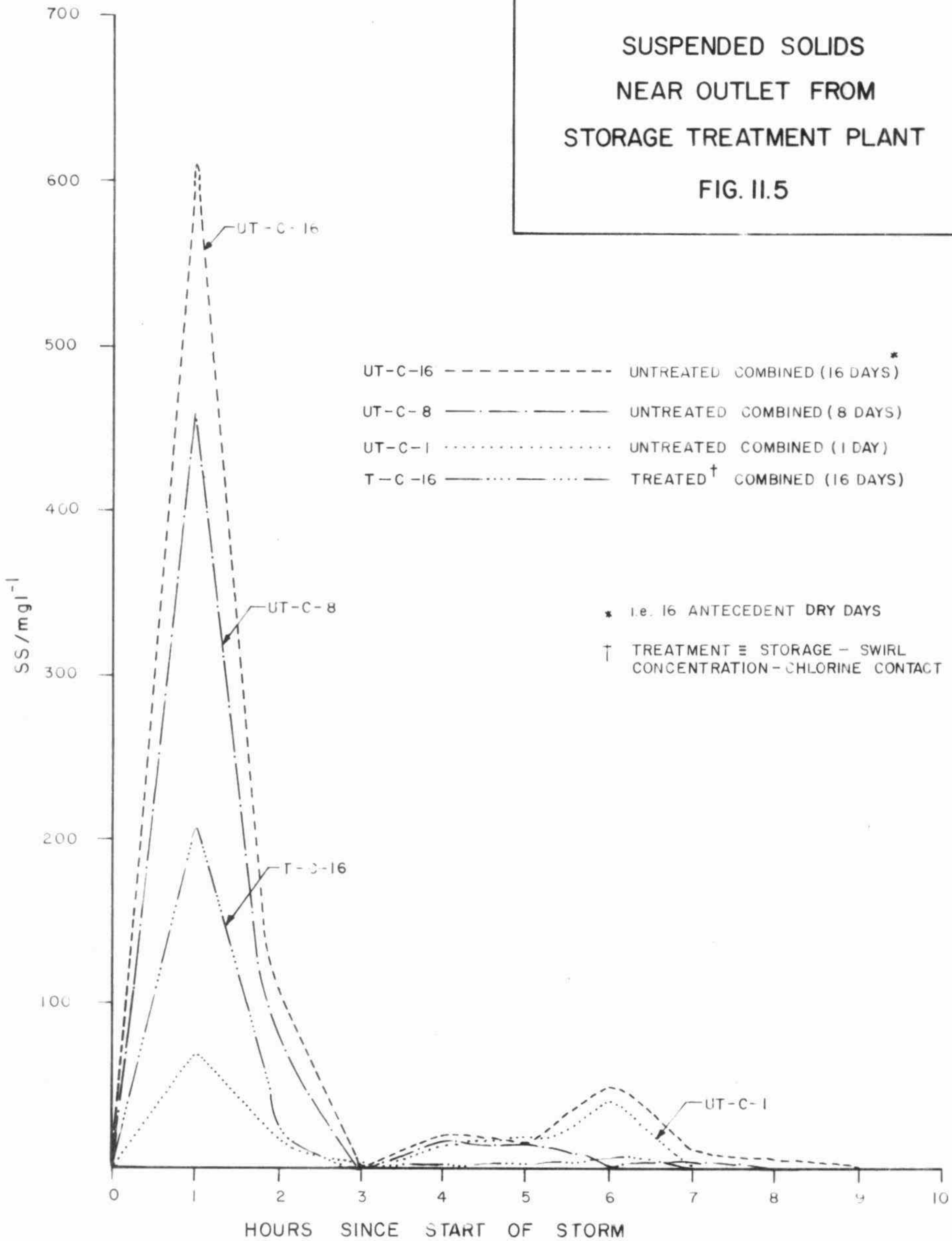
SS FROM TRANSPORT TO TREATMENT SYSTEM

(SEPT. 5, 1971 STORM)

FIG. 11.4

SUSPENDED SOLIDS
NEAR OUTLET FROM
STORAGE TREATMENT PLANT

FIG. 11.5



DEVELOPMENT OF A METEOROLOGICAL DATA ANALYSIS PROCESSING PROGRAM

CHAPTER 12

CHAPTER 12
DEVELOPMENT OF A METEOROLOGICAL DATA ANALYSIS PROCESSING PROGRAM

12.1 GENERAL

The meteorological data requirements of both the SWMM and STORM models may be quite extensive, depending upon the number of events simulated and the degree of modelling detail. The STORM model usually employs several years of hourly precipitation and temperature as a data base, while the SWMM requires short interval (often 5 minutes) rainfall data for each event simulated. Since the SWMM may also be used in with a larger time step "lumped" fashion for the simulation of many events, organization of large amounts of meteorological data will often be necessary. The large amount of these data creates opportunities for both random and systematic errors which may lead to poor simulation and greater de-bugging efforts.

The Terms of Reference for this study required that subroutines be written for the creation, and management of a data base for modelling purposes using data from existing data banks. The most important source of meteorological data for real storm simulation is the Atmospheric Environemnt Service (A.E.S.) [1]. This chapter describes the specific routines which form the Data Analysis Program. This is intended to provide an interface between the urban runoff model, requiring data in a specific format, and the data bank, in which information is stored in specific formats.

12.2 METEOROLOGICAL DATA PROCESSING IN CANADA

Climatological data have been machine processed in Canada since about 1950. Prior to this, all recorded data were reduced by clerical methods. The basic data input medium is now the punched card, which is created from recorded data forms. Punched cards are computer processed and the data is recorded on rapid access disc or storage devices, while the cards are

retained as a secondary source of information. A.E.S. holds virtually all climatological data collected in Canada.

12.2.1 *Maintaining the Data Bank*

The four principal activities [2] in monitoring the data bank are:

(a) Generation of punched cards
(b) Quality control
(c) Processing
(d) Storage

Both manual and automatic methods are used to produce punched cards containing the information recorded on data sheets. The majority of the 350,000 new cards produced per month are punched manually, although direct automatic generation of cards from experimental gauges is being tested at a number of stations. Quality control programmes [3] have been developed to check the data stream for accuracy, completeness and random errors. Transcription errors, discrepancies and missing data are identified and the pertinent records completed manually. At present the homogeneity of the data, for a single gauge or between gauges, is not assessed. Consequently the user must perform mass analyses and similar checks to establish the suitability of a particular record or group of records. The five operations carried out by the A.E.S. for the identification of erroneous data are:

(a) The data are scanned and suspect records are isolated for further action.

(b) These data are examined in order to justify their accuracy if possible.

(c) Where accuracy cannot be justified an attempt is made to determine the incorrect procedure used by the observer and to correct the data accordingly.

(d) When the remedial measures outlined in the two previous steps fail, the recorded value is rejected and is not included in further processing.

(e) At this stage, estimated data are entered in lieu of rejected or missing records depending on the type of data and the necessity for completeness.

The edited record is stored on disc storage devices and eventually transferred to the archives, usually in the form of punched cards, although the actual discs are being used increasingly for ultimate storage. Data is recalled from the archives when required for the preparation of reports, tabulations and distribution files to satisfy requests from individuals or organizations. In spite of these quality control operations, some errors do occur in A.E.S. supplied data.

12.2.2 *Data Formats*

Twenty-one different data formats are presently used by the A.E.S. These are referred to as "card types" because of the cards onto which all data is originally punched. All the "card types" are described in Volume 2, Appendix 6 of this report. Card types 3 and 4 contain the hourly precipitation and temperature data required for urban runoff modelling [4,5]. Shorter interval precipitation data have have not been abstracted and digitized into the bank at present. These rainfall records must be obtained from photocopies of the original tipping bucket records for the desired storms. All data can be obtained from the Environment Canada A.E.S.

12.3 THE DATA ANALYSIS PROGRAM

The Data Analysis Programs have been developed to accommodate meteorological data from the A.E.S. data bank, and to create and manage the user data base for the SWMM and STORM models. The three basic functions involved are:

(a) Reading the data from the A.E.S. bank and screening for faulty data.

(b) Analysis of the data for consistency and suitability, and combination of the records of several gauges.

(c) Creation of a card data base in the appropriate format for the models.

Figure 12.1 shows the functional flow chart for the Data Analysis Program.

12.3.1 *Reading and Screening*

The quality control procedures employed by A.E.S. have to be supplemented by a data screening process before the creation of the user data base for modelling. This function is indicated as block "A" in Figure 12-1. Over-punches are employed by the A.E.S. to denote special conditions, such as freezing precipitation. When these are encountered, appropriate messages are printed and the data is replaced with a zero. In this manner unintelligible or missing data, which would be rejected in subsequent modelling, are removed. In addition, the user may choose to replace the rejected data with more realistic values. Frequently gaps occur in the precipitation record due to gauge or recorder malfunction. In the absence of card type 3 hourly rainfall data, the Data Analysis Program abstracts the 6-hour precipitation totals from card type 4 and distributes these equally throughout the missing period. (This technique is described in more detail in Volume 3 of this report). The resulting precipitation record may be subsequently modified using techniques described in the following section. These more precise techniques should be employed when the period of missing data is significantly long, such as those resulting from winter shut down of certain gauges. Raw data are screened to identify occasions on which temperature and precipitation values fall outside user defined limits. Simple plots may be requested to facilitate visual inspection. These methods allow rapid detection of any random errors present in the data.

12.3.2 *Analysis and Processing of the Data*

Following the preliminary screening it may be desirable to combine the records of several gauges to produce a record more representative of some intermediate location. This function of the Data Analysis Program is shown in Figure 12-1, block "B". Combination of records may be accomplished on the basis of the "normal ratio" method, the "Theissen Polygon" method, or by employing some weighting factor based upon the distance or the gauges from the study area. The consistency of long periods of data or the individual gauges should be investigated before combination. Single or double mass analysis techniques are available to facilitate these investigations and to determine the significance of variations in the relative catch of a gauge over the period of record. A summary analysis of the edited and combined precipitation record is generated by the model. This is useful for identification of critical events and for establishing initial conditions for one event simulation.

The capabilities provided in the Data Analysis Program may be summarized as follows:

(a) Combination of Gauge Records - A variety of techniques may be employed to combine different gauge records for the purposes of estimating missing data or constructing a weighted average precipitation record for a specific location. The basic computations are performed using the general formula:

$$P = \frac{P_1X_1 + P_2X_2 + \dots + P_nX_n}{\sum_{i=1}^{n} X_i} \qquad (12.1)$$

where P is the combined precipitation

$P_i \ldots P_n$ are the individual gauge precipitations

$X_i \ldots X_n$ are weighting factors

The weighting factors (X_i) are user supplied and may be computed by the technique appropriate to the desired purpose. Both hourly and short interval records may be processing in this manner.

Missing precipitation data may be estimated from the records of three or more neighbouring raingauges using the "normal ratio" method. The long-term precipitation totals at the particular stations, including the station for which the record is to be generated, are computed manually and the ratio of the normal at each station to the normal at the subject station is used to determine the relative contribution from each record. The weighting factors applied to each record are computed from:

$$X_i = \frac{Ne}{Ni} \qquad (12.2)$$

where Ne is the long-term normal at the gauge for which the estimate is required

Ni is the average long-term normal of the surrounding gauges

The average rainfall record for a specific area may be estimated from the records of neighbouring gauges. The general combination formula in the Data Analysis Program may be used to compute the average precipitation record by either the "Theissen Polygon", or "arithmetic average" method. For each case the appropriate gauge weighting factors are computed separately.

(i) <u>Theissen Polygon Weighting</u>

$$Xi = \frac{Ai}{At}$$

where Ai is the area of the polygon associated with each gauge

At is the total catchment area

(ii) Arithmetic Weighting

$Xi = 1$ (i.e., all gauges have the same weighting)

(iii) Distance Weighting

$Xi = Li$

where Li is the distance from each gauge to the centre of the study area

(b) Mass Analysis - Changes in gauge location, exposure, instrumentation or manual recording procedures may result in variations in the measured precipitation catch from a particular gauge. Such changes are often difficult to detect and not analyzed in published records. Double mass analysis may be used to check the consistency of a gauge record by comparing its accumulated precipitation record with the concurrent accumulated average values for several surrounding stations.

A change in the relative catch of a station may be observed when the accumulated records are plotted. The Data Analysis Program has been formulated such that a double mass analysis may be performed for any selected input station record and a plot of the resulting curve be produced for visual inspection.

(c) Event Summaries - It is often useful to pre-process the precipitation record before modelling in order to define the antecedent conditions for specific events. The Data Analysis Program allows the user to supply a storm definition in terms of the minimum amount of hourly rainfall necessary to signal the start of a storm event and the number of dry hours signifying the end of that event. Thus the program can identify the storms within the given period of record and print out summary statistics for each. In some cases this initial analysis is sufficient for identification of critical periods for subsequent detailed

modelling. The event summaries consist of information on:

- storm index number
- start time, end time and duration
- antecedent rainfall
- peak rainfall intensity

12.4 CONCLUSIONS

(a) A Data Analysis Program suitable for the construction and management of a user data base suitable for both single event and continuous simulation has been developed. The program is operated independently of both the SWMM and STORM models to provide the necessary means for organizing the large volumes of meteorological data required for running these models.

(b) The program is simple to operate and satisfies an obvious practical need. The program can process short interval data as well as hourly records and with simple output format changes may be used to generate precipitation data for any urban runoff model.

REFERENCES - CHAPTER 12

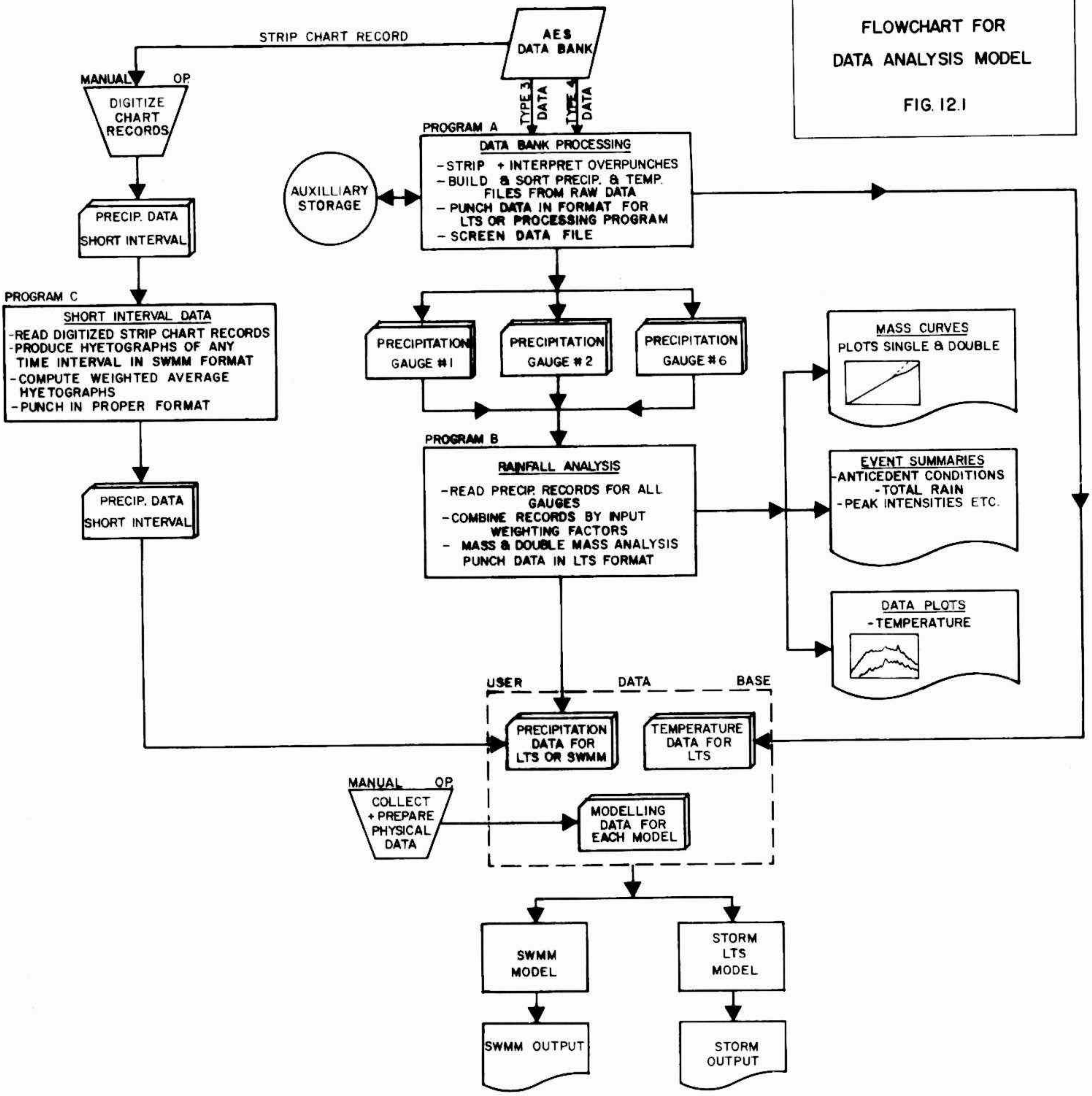
1. Cudbird, B.S.V., "Modern Techniques in Canadian Climatic Data Processing", Department of Transport, Meteorological Branch, 1968.

2. Potter, J.G., "Quality Control of Surface Meteorological Data for Cliamatological Purposes", Department of Transport, Meteorological Branch, 1969.

3. Atmospheric Environment Service Notes, "W-3-A Quality Control", File 1684-8, December 1969.

4. Department of Transport, Meteorological Branch, "Card Type 4-Tape Format Information", 1970.

5. Department of Transport, Meteorological Branch, "Card Type 3-Revised Card Format Documentation, 1969.



FLOWCHART FOR

DATA ANALYSIS MODEL

FIG. 12.1

GENERAL CONCLUSIONS AND RECOMMENDATIONS

CHAPTER 13

CHAPTER 13

GENERAL CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

(1) An analysis of the available measurements of stormwater runoff events and combined sewer overflows on urban watersheds in Canada indicated that no one set of data is perfect for a general validation of all SWMM routines. The precipitation and flow measurements from three watersheds, of 48, 542 and 2330 acres were found to be acceptable for comparison with the SWMM quantity simulations for those areas. Pollutographs were compared for both a combined and a storm sewer system. Additional information from past studies, and independent studies conducted in parallel to the present work, was also considered.

(2) The comparison of measured flows with those simulated by the SWMM over a wide range of watersheds of varying size and characteristics confirms that the model (with the appropriate level of discretization) predicts peak flows, time to peak and total volumes sufficiently accurately for planning applications and for design work (when surcharge is not required).

(3) The sophisticated WRE TRANSPORT routine, which is being incorporated in the SWMM, and the Dorsch HVM model give superior simulations to those resulting from the use of the initial SWMM TRANSPORT routine for surcharged sewer systems. The results of simulations for free surface flow obtained from all three models are considered equivalent. The WRE and HVM routing models were compared for similar systems using identical input flows; the results of the two models are considered to be equivalent from a practical viewpoint. The inlet hydrographs generated by a detailed HVM simulation and a less detailed SWMM simulation were very similar.

(4) The analysis of different urban runoff quality models indicates that the SWMM incorporates the most comprehensive quality routines of those reviewed. However, the accuracy possible in quality modelling is not comparable with that obtained in quantity modelling, due to the greater complexity of the phenomena involved.

(5) The SWMM quality model, when calibrated, gives estimates of the order of magnitude of peak pollutant concentrations, and of the total pollutant load released to the receiving water, during a stormwater discharge event.

(6) Methods resulting in significant simplifications of both the quantity and quality simulation procedures have been developed for the computation of catchment outfall hydrographs and pollutographs. It has been shown that, if these methodologies are adhered to, large urban watersheds can be modelled in an aggregated manner with a minimum of input data, without a significant loss of accuracy as compared to equivalent detailed simulations.

(7) A Generalized Quality Model based on the equations used in the SWMM has been developed and documented. This model requires less data preparation and computer processing time than the SWMM and may be more rapidly calibrated than the corresponding SWMM routine. This model may be used in conjunction with flow simulation models or with recorded flows.

(8) Snowmelt quantity and quality sub-models have been integrated with the SWMM as User Options. These models have been tested against preliminary field data.

(9) All of the other SWMM routines have been tested and debugged and subsequently assessed and/or modified. In particular, the storage/treatment cost routine was adapted to include cost information based on Canadian data.

(10) Comparison of the results of STORM with those of detailed SWMM simulations and with measurements emphasize that the very simple STORM model is an excellent tool for predicting the quantity and number of individual overflows from a particular district.

(11) A new model for the processing and analysis of meteorological data for use with STORM and SWMM has been developed and documented.

(12) The interfacing of the models investigated has been discussed, and a package of models capable of simulating all aspects of stormwater flow and quality, from the planning to the design phase, has been presented. Particular emphasis has been placed on the applicability of the package to Canadian conditions, as well as on the level of sophistication appropriate to the different stages of a study and on the selection of the "right model for the job".

RECOMMENDATIONS

(1) SWMM flow simulation has proven to be accurate in a large number of applications for areas of greatly differing size, land use and sewer system configuration, and over a range of meteorological conditions. The incorporation of the WRE TRANSPORT routine will extend the capabilities of the original SWMM to include the analysis of surcharged systems. Consequently, it is considered that the use of the SWMM can be recommended for the simulation of flows, where required, in most drainage planning and design studies, although particular advantages offered by other models in certain specific situations should not be overlooked.

(2) The use of STORM is recommended in planning studies for simulating the volume and number of overflows over the period of the long-term meteorological record. STORM should be used for the generation of

a statistical analysis of the effects of storage on reducing overflow volumes and for an approximate guide to the probable associated reduction in the total pollutant loads discharged. STORM is particularly useful for the screening and comparison of alternatives and for the isolation of critical events for subsequent SWMM simulations.

(3) At present, the modelling of runoff quality cannot be expected to be as precise a science as the simulation of runoff quantity. In some specific situations different from those examined in the report, a limited number of flow measurements may be required for quantity simulations. However, it is considered that some sampling of the quality of runoff and overflows will usually be required to supplement the final modelling activities in a study directed towards the design of pollution abatement facilities. Past experience with the interpretation of the results of sampling programmes, and subsequent efforts for calibration, has emphasized that the collection of samples should be very carefully planned in view of the high costs involved. Preliminary modelling is often useful in determining the specific sampling locations required.

(4) Simulation of stormwater quality will be useful in a variety of studies such as the assessment of the requirement for upgraded treatment plants, the need for new plants, the benefits of sewer separation or new interceptors and the control of runoff from new developments.

The costs of treatment and to a lesser extent, storage facilities, are partly a function of the design flows. Although the modelling of quality is by no means exact, accurate flow simulation is very useful for the design of any pollution abatement facility. Even if the calibration of the parameters controlling quality is limited, quality simulation can still be of value for illustration of the severity of a problem and comparison of alternative pollution control schemes. Consequently, it is recommended that quantity and quality modelling form an integral part of all studies for pollution abatement.

(5) Any well validated computer model that has proven suitable for specific location conditions will be a valuable tool and decision. The particular advantages of the SWMM are its wide availability, the excellent training programmes offered, and, most importantly, the potential for the study of <u>both</u> quantity and quality throughout the urban system and in the receiving waters.

Implementation of modelling techniques will be most successful if the advantages and limitations of different models are understood. The interfacing of models outlined in this report, such as the complementary use of STORM and SWMM is considered an example of the hierarchical use of models for different stages of a project.

FURTHER WORK

It is considered that additional research is needed in the following areas:

(a) Additional efforts and research for the improvement and refinement of existing models.

(b) The further accumulation of data to facilitate these improvements.

(c) Research into, and documentation describing, the range of pollution abatement and drainage design alternatives.

The majority of past research has been concentrated in the first category; that is, the tools have been studied to a greater extent than the problems. However, there are definite needs for further work in the first two categories, the improvement of models and the extension of their capabilities. These include:

- Development in the SWMM of a routine to account for the water balance betwen storm events to permit use of simplified versions of SWMM for continuous simulation.

- Refinement of the WRE model to account for a wider variety of sewer system structures and conduit shapes.

- The formulation and testing of new relationships for the simulation of quality in the SWMM RUNOFF and SNOWMELT routines.

It is considered that significant improvements in the approach to quality modelling will only result from the acquisition of substantial amounts of new sampling data from a large number of areas of diverse characteristics.

The extension of the uses of the models investigated in this report for a number of additional studies is a most promising area for further research. Some examples include:

- The use of the SWMM for continuous simulation (this work is being pioneered by the University of Florida).

- The application of the SWMM for rural basins, or for areas with a low level of urbanization. This work would eventually lead to uniform methodologies for all flooding studies and would assist in the more rigorous analysis of the effects of urbanization.

- The use of the SWMM for the prediction of inflow into sanitary systems, and in general, for the more accurate simulation of the hydraulic loading on treatment plants.

Future research into the solutions to specific problems will be more productive from the practitioner's view point, than efforts directed towards the improvement of the available tools. For instance, the existing versions of the models currently available may be used without additional changes for:

- The examination of new drainage policies and optimum methods for flood control.

- The assessment of established design criteria, such as the design storm frequency and the design storm itself.

In addition, existing models may be used in the quantification of problems of broad environmental concern, such as:

- The relative importance of different sources of pollution, non-point sources, rural drainage, urban runoff etc.

- The most suitable locations for, and patterns of, new developments at a municipal and regional scale.

The widespread acceptance and application of urban runoff models within the profession will eventually be better established by the savings resulting from modelling techniques, and the achievement of well defined policies rather than by the exaltation of the theoretical merits of particular models.

IMPLEMENTATION ON COMPUTER

The computer tape presented as part of the final report on this study contains the Storm Water Management Model System in a version which will run under IBM Fortran IV Version H.

All system's development work was done on one machine and under one compiler, which was most appropriate, in view of the fact that considerable effort was required to debug the various programs before effective testing could begin. To have done this on several different computer systems would have been an expensive task.

The conversion of a system as large as this for use on other computers, will require a very considerable effort for each new machine; (the term "compatible Fortran" is a relative one).

If the Scientific Authority wants to improve SWMM as new studies may require, the problem of maintaining different versions on different computers could be very considerable. Problems of portability, compatibility, and maintenance are now being affected by two technological advances, namely in terminal

operations, and in communications capabilities. By implementing the system on one central computer and permitting people to access it via telecommunications, the problems described are almost wholly obviated.

The system as it now exists is in a form which will be readily available to most engineers or organizations who wish to use it, since it will run on the most common large commercially available computer, the IBM 370 series. For example, Datacrown, the Toronto data centre used for most of the computer work on this study, has, or will have within the next few months access to its Toronto computer via terminals in all of the Provincial Capitals, giving access to the Toronto system with no long-distance telephone charges.

In view of the above, it is our recommendation that the SWMM Model not be implemented in different Fortran versions at this time. Rather that it be made available in a method similar to that described above in the version prepared in the study and that consideration be given in the future, if it seems appropriate, to conversion to other machines. This will permit close control over the model in the early stages of use across the country and permit dynamic modifications to be handled with least effort.

If the Scientific Authority so desires, we will be prepared to submit a proposal for conversion to named competitive large scale computer systems, on a time basis.

Should it occur that pressure is exerted from commercial data processing houses with such competitive equipment, for the use of the system on their machines, it may be appropriate to make the present version available to them at no cost on the understanding that they be responsible for conversion to their own system. They might then provide a service to users charging for use of their system, but not applying any surcharge for use of the program. Alternatively, some royalty system might be devised dependent on the manner in which the government wishes to make the system available.

This would ensure that the conversion was done by people most familiar with the specific computer system involved. It would ensure that the decision to incur the cost of conversion would be based on market potential, and finally it would permit the conversion to be done at no cost to the government.

The relative merits of batch processing with normal turn around in the Toronto area, and interactive time sharing considering the various phases of Urban Watershed Modelling were also considered.

Some definition of terms is necessary, since usage throughout the industry varies somewhat. For the purposes of this study, batch processing is defined as a process in which a run is submitted to the computer either through a high speed device such as a card-reader or through a low speed device such as a keyboard terminal,and the job is then run and the output returned to the user either immediately or at some later time, with no communication between the user and the program from the time of submission to the receipt of results. Interactive computing is defined as a process which may be initiated in either of the ways described above for batch processing, but in which the program will halt at certain pre-determined points to permit the user to make certain decisions before the run is continued. One further term should be noted, conversational computing. This is similar to interactive computing, but specific prompting questions are written into the program so that a psuedo - conversation can be carried on between the program and the user in an effort to prompt or guide the user in the making of the run time decisions. The decision as to whether batch or interactive computing should be used depends on two issues principally, that of the most appropriate method of use for the engineer i.e., whether or not it is important for him to be able to interact or converse with the program in terms of the design function which he is attempting to carry out, and the amount of data which is required to set up the various runs of the system.

Regarding use of SWMM, the engineering user does not seem to have a great need for interaction, seeming, in our testing, to operate perfectly adequately in a batch node. The amount of data required to set up the runs of the SWMM Model are very large and unsuitable for entry via a keyboard terminal. With the system as it now exists, it is possible to access the program in an interactive mode via keyboard terminals using existing control software owned by for example the commercial data centre, Datacrown, used for the development work. This could permit an engineer to set up a major run in a batch mode, and then having completed a first run modify certain input

parameters via a keyboard terminal, retain the rest of the data unchanged, and initiate another run.

To permit conversational computing would require major changes to the SWMM Model itself, and although such a process is most helpful for the neophyte user, it is not considered that it would be worth the effort to implement such a system at this time.

Accordingly, it is our recommendation that batch processing continue to be the prime mode of operation for the system.

ACKNOWLEDGEMENTS

This study was sponsored by the Storm and Combined Sewers Sub-Committee within the framework of the Canada-Ontario Agreement. The scientific authorities for the project were J. Marsalek (main contact), Dr. R. Slater and G.H. Mills. The project steering committee - J. Marsalek, C. Howard, H. Torno, and D.H. Waller provided close guidance and frequent reviews throughout the entire project.

The team responsible for this study comprised personnel from Proctor and Redfern Limited (PR) and James F. MacLaren Limited (JFM). The key members and their roles were as follows:

Mr. P. Hertzberg	(PR)	-	Project Manager
Dr. P. Wisner	(JFM)	-	Technical coordinator
Mr. H. McGrory	(PR)	-	Computer coordinator
Mr. A. Brodie	(JFM)	-	Assistant project manager
Mr. A. Perks	(PR)	-	Assistant technical coordinator, Co-editor
Mr. H. Belore	(JFM)	-	
Mr. F. Lee	(PR)	-	
Mr. A. Roake	(JFM)	-	Co-editor
Mr. P. Cox	(PR)	-	Computer Systems
Dr. N. Zaghloul	(JFM)	-	
Mr. L. Pataky	(PR)	-	
Mr. M. Ahmed	(JFM)	-	
Mr. P. Dick	(PR)	-	
Mr. W. Clarke	(JFM)	-	

Several other staff members provided special inputs or assisted in preparation of data.

Dorsch Consult also participated in the team and was responsible for the Dorsch HVM simulations presented in the report. The assistance of Mr. F. Mevius was appreciated in this regard.

Water Resources Engineers kindly made the latest variation of the WRE model available to the team. The cooperation of Dr. R. Shubinski, Vice-President of WRE, is gratefully acknowledged.

The team greatly benefited from discussions with, and assistance from, several individuals and consultants as follows:

Prof. S. Solomon	University of Waterloo
Prof. G. Ganczarczyk	University of Toronto
Prof. E. Watt	Queen's University
Mr. C. Kitchen	City of Toronto Public Works Dept.
Mr. G. Burns	City of Winnipeg
Mr. L. Roesner	W.R.E.
Mr. J. Peters	U.S. Corps of Engineers
Mr. J. Abbot	U.S. Corps of Engineers
Dr. W. Huber	University of Florida
Dr. J. Heaney	University of Florida

The team sincerely acknowledges the contribution made by all of the above-named personnel and also that made by many others, too numerous to list.

The extensive typing work was performed with dedication by Ms. Annette Di Nallo, to whom the Study Team extends their appreciation.